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MAR 8 1917

The Miami Valley Flood- Protection Work

Five Articles published in Engineering
News during January and February 1917

- I. Fixing Maximum Flood Limits
- II. Why Retarding Basins Were Adopted
- III. Dam and Outlet Problems
- IV. Study of Retarding-Basin Operation
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The Miami Conservancy District
Dayton, Ohio
1917

The Miami Valley Flood-Protection Work

I—Fixing Maximum Flood Limits

A Pioneer Study of Storm Records in the United States; Analysis of Storm Frequencies, and Prediction of Future Storm Possibilities in the Central Region; The Final Result, Ten Inches Runoff in Three Days

After the terribly destructive flood of Mar. 23-28, 1913, the people in the valley of the Great Miami River, Ohio, made an urgent appeal for protection against future floods. This put a wholly new problem before the engineers who were called in to devise such protection. They had to determine what future great floods might visit the valley; or, still more difficult, what is the *greatest* flood that may come. No such forecasting had ever been done for a stream and a drainage area like the Miami, or for intense rainstorms covering thousands of square miles.

While this problem may seem like one of pure prophecy, it was actually solved by means of a monumental research into rainfalls and floods. Records were fully analyzed, and their facts extended into the future by an interesting combination of judgment and calculation. From an examination of this work, it can be asserted with all confidence that the first premise of the great Miami flood-protection enterprise rests on a thoroughly sound basis.

The research is of entirely new nature. Similar work will undoubtedly be needed elsewhere in the future. Fortunately, many of the data collected in the present research are applicable to other regions; but some of them are valid for the Central States region only, and possibly are even more limited than this. The distinction between the two classes of results must be carefully observed.

THE STARTING-POINT OF THE WORK

To state the final results in advance, Figs. 1 and 2 are given. The former shows the Dayton plain in flood in 1913, and as it would be flooded by the ultimate maximum flood. It also shows how both floods are to be controlled by the retarding basins, which form the central feature of the flood-protection works soon to be built.

Fig. 2 gives the adopted flood-flow curve, the runoff of the ultimate greatest storm covering the entire valley, as used in designing the Miami plan. Comparison of the two upper lines of the intensity diagram with the dotted line, showing the 1913 intensity, expresses clearly what excess is provided for. The same appears from the diagram of accumulated flow, the right-hand diagram in Fig. 2. For better understanding of the two diagrams, the rate of flow is expressed not only in inches depth, but also in second-feet per square mile. In the accumulated-flow diagram the retarding-basin capacities (to spillway level) are represented, showing how little is left for the stream channels below, even in the greatest flood.

Though the 1913 flood, which overtopped the levees at Hamilton and Dayton by ten or a dozen feet, was greater than any Miami flood in living memory, yet it was taken as a storm that undoubtedly must be protected against. Further, the engineers brought with them knowledge of some of the greatest storms recorded in the Central States. The largest of these, the storm of October, 1910, centering near the mouth of the Ohio River, in Illinois, Kentucky, Tennessee and Missouri, was typical of summer or

fall storms in having a low ratio of runoff; yet in total amount of rainfall it was about 45% greater than that of 1913 for an area equal to the Miami watershed. It was kept in mind as a provisional maximum, while detail investigations were being made.

There was no lack of popular belief that such a flood as the one of 1913 could not happen again, that it is not in the regular order of natural phenomena, but represents a cataclysmic happening. At the same time it developed that even men in close touch with weather and flood matters were not fully informed as to great storms. Under the circumstances the engineers felt it vitally necessary to look up storm records in the most thorough manner possible, with a view to supporting or modifying their views by the results of study of past

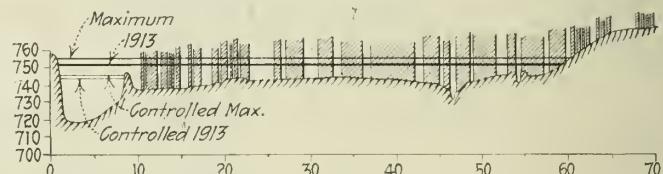


FIG. 1. CROSS-SECTION OF THE DAYTON PLAIN Flood of 1913, with actual possible and controlled flood stages

flood history and estimate of future possibilities. Such a research was at once entered upon.

Confirmation of the starting-point ideas was obtained in a very short time. On bringing together the historical records of the Miami Valley and reducing the vague data of 70 and 100 years ago to figures as well as trained judgment would permit, two facts came out: First, since the coming of the white men there has been no Miami flood equal to that of 1913, not merely in gage-height and extent of inundation, but also in volume. Second, indications are that one or two prior floods came within reaching distance of 1913. The 1913 flood, while considerably greater than earlier recorded ones, is not so pre-eminently ahead of them as to constitute a cataclysmic phenomenon. Rather, it forms a natural and harmonious element of a progressive list, though for the Miami it indisputably stands at the top of the list. This latter fact was modified and made very prominent by the subsequent researches, which examined conditions outside the Miami Valley.

GREAT STORMS AND FLOODS IN THE PAST

Two points had to be determined by the research: What storms and floods have happened in the past, and what are to be expected in the future. No physical evidences that gave help in the matter were found in the valley. The limited period of the records for the Miami needed extending by the aid of records from other regions.

Studying the records available at Dayton and in the Columbus office of the Weather Bureau proved insufficient.

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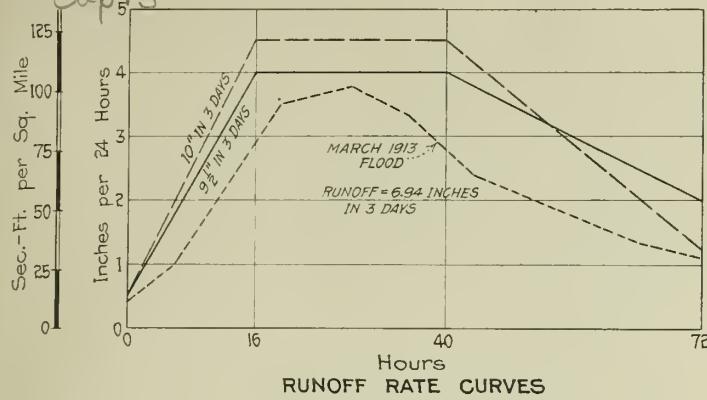


FIG. 2. RUNOFF AND ACCUMULATED RUNOFF CURVES FOR THE MIAMI VALLEY; THE MAXIMUM POSSIBLE, AND THE 1913 FLOOD

The capacities of the five retarding basins are represented by the straight lines in the right-hand diagram; the ordinate at zero hours represents the initial capacity, while the other ordinates represent initial capacity plus conduit discharge.

Therefore several men were sent to Washington and spent eight months there, abstracting the entire storm rainfall records of the country.

The Miami area, 120 mi. long, northeast to southwest, by 50 mi. wide, containing 4,000 sq.mi. (exclusive of the tributary Whitewater near the mouth), lies in 40° to 41° latitude and at 500 to 1200 ft. elevation above sea level. Its climatological character is the same as that of the whole Ohio River basin, and experience throughout the basin is therefore pertinent testimony.

Storm Abstracts—The rainfall records of all storms in the weather records of the United States east of 103° longitude were abstracted. All rainfalls were noted down that exceeded the following limits: 1-in. precipitation in 24 hr., or 4 in. for the total storm, where the normal annual rainfall is below 20 in.; and 10% of the annual total in one day, or 15% of the annual total in the whole storm, for stations whose normal annual rainfall exceeds 20 in.

Each station (some 3,000 observing stations were included) had a separate sheet, where besides the daily rainfall were given for each separate storm the one-day total, the two-day and three-day accumulations and sometimes four-, five- and six-day accumulations.

Districting the Country—For easier orientation the whole country (east of 103°) was now districted into 2° quadrangles (divided on the odd degrees); they numbered 133. Designating the rows by numbers and the files by letters, each quadrangle (as 9 E for the Miami) could readily be recognized with respect to its neighbors. Stations on each quadrangle were designated by a, b, c, etc., progressing southwest from the northeast corner. Thus, station 9 E z is in the neighborhood of 10 F a. Marking the station sheets accordingly, ready grouping of the storm rainfalls of separate stations into single storms was possible.

A "storm index card" was made for each storm—about 3,500 in all—by bringing together the corresponding precipitations shown by adjoining stations, as traced out through the system of designation just described. This paved the way for drawing storm maps, to furnish a basis for time-area-depth studies.

TIME-AREA-DEPTH STUDIES OF STORMS

Half of all the storms indexed were found to be one-station storms. Two- to six-station storms amounted to 35%, while 15% covered more than six stations. The latter received nearly all of the subsequent study.

It is necessary to remember that the point of importance now was not the greatest rainfall intensity, nor the greatest 24-hr. precipitation at one station, but the

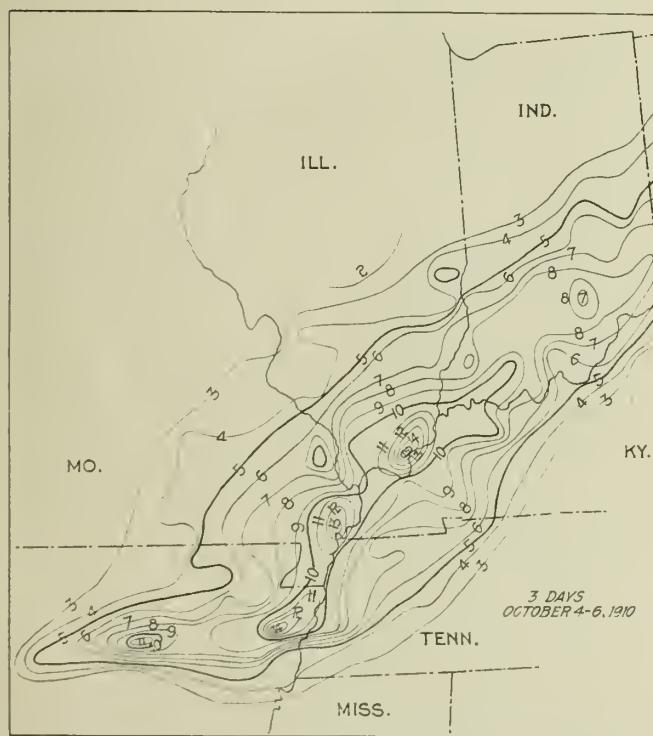


FIG. 3. THE GREAT STORM IN SOUTHERN ILLINOIS, OCT. 4 TO 6, 1910; THREE-DAY RAINFALL DEPTH CURVES

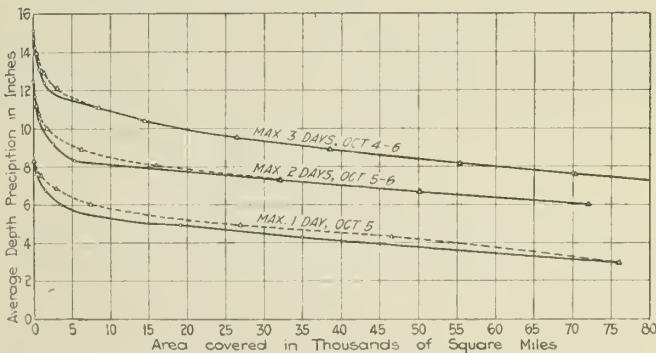


FIG. 4. TIME-AREA-DEPTH CURVES OF RAINFALL FOR SOUTHERN ILLINOIS STORM

greatest accumulated mass of precipitation over a large area during a period of several days. Storms had to be compared on the basis of the three factors of area of storm, duration, and accumulated depth of rainfall. Drawing contour maps of accumulated rainfall was found

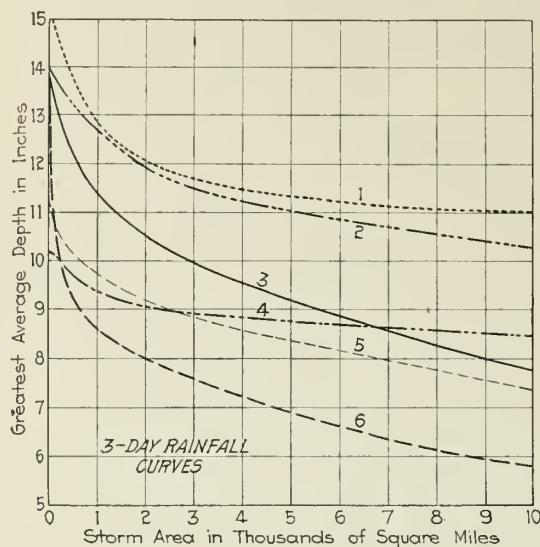


FIG. 5. THREE-DAY AREA-DEPTH CURVES FOR SIX GREATEST STORMS IN THE CENTRAL BASIN

1—Southern Illinois, Oct. 4 to 6, 1910. 2—Arkansas, Aug. 17 to 20, 1915. 3—Iowa, July 14 to 16, 1900. 4—Ohio, Mar. 23 to 27, 1913. 5—Kansas, July 5 to 8, 1909. 6—Michigan, July 19 to 22, 1909.

to be the best means of deriving the curves needed for the comparison.

Only storms covering at least 500 sq.mi. and with maximum intensity of at least one-fifth the total annual rainfall were selected for mapping. Of 46 such storms, the 27 greatest were finally chosen. The nature of the maps is illustrated by Fig. 3, showing the southern Illinois storm of Oct. 4 to 6, 1910; the three-day map is given, those for other lengths of time being similar in general appearance.

The time-area-depth curves for the same storm are shown by Fig. 4, which includes the curves for one to three days. Similarly, Fig. 5 shows in grouped arrangement the three-day area-depth curves for six storms, of which five occurred north of the Ohio River and east of the Dakotas. These (and three others that reached a limit at two days) are the great storms of the Ohio River basin, a region which is a unit as concerns factors of climate and storm incidence. Three of them were much heavier three-day storms than that of 1913¹, and these obviously were possible storms for the Miami Valley.

The storm records showed the general fact that the maximum intensity does not occur at the start of a storm, nor at the end. The heaviest day's rainfall in any storm was found likely to be about half the total of the storm. On the basis of these facts the general shape of the flood-runoff curve (Fig. 2) already began to outline itself.

LONG-TIME RIVER RECORDS

The storm records thus compiled cover only a moderate period of years, but a sufficient length of time to include several very exceptional occurrences. The question of whether the possible maximum could perhaps be *very*

¹The 1913 storm totaled 7.6 in. on the Miami area, but was heavier a little to the west. The maximum fall on an area as large as the Miami, but lying partly outside, was 9 in. in three days.

much higher—that is, whether a wholly abnormal great storm might occur some time—was cleared up by a study of old river-flow records.

Ohio River stages at Cincinnati are recorded for 130 years. The Seine at Paris has records for 300 years, the Danube at Vienna for nearly 1,000 years, and the Tiber at Rome for 2,000. In none of these records was any indication of periodicity found. They tell, further, that the greatest flood in a thousand years does not very largely exceed the flood of a magnitude recurring about once in 50 years.

All great floods come within a few feet of each other, and “there has been no great exceptional flood that brought down half again or twice as much water as the other great floods.”

The conclusions that could be drawn from the facts developed were expressed very concisely by Chief Engineer Arthur E. Morgan in his testimony before the Conservancy Court a few weeks ago:

The 1913 flood was one of the great floods of centuries, but in the course of three or four hundred years we may get a flood 15 or 20% greater. We think that is possible, but do not believe we will ever get a flood greater than 20 or 25% in excess of the 1913 flood. But as stated before, we have a factor of ignorance against which we must provide, and the only way we can do this is by arbitrarily increasing the size of the maximum flood provided for. If we had longer records, we could estimate closer; but we believe that in planning works on which the protection of this valley depends, we must go beyond our judgment in the matter. We have done this on every other phase of the design, and we believe that it will not be good engineering practice to stop at our judgment on this phase. We must be able to say that our engineering works are absolutely safe in every respect, and for this reason we have gone 15 to 20% beyond what we believe to be the greatest possible storm, and have provided for one 40% greater than that of March, 1913.

To secure further evidence bearing on the conclusions, the storm records already compiled were analyzed to get the average time-interval of storms of a given size.

TIME-INCIDENCE OF STORMS AND THE MAXIMUM LONG-TIME STORM

The underlying idea of the frequency studies was that the storm experience of one station or one quadrangle adds to that of others. The total experience-period, therefore, is the sum of the separate experience-periods. In detail, two methods were pursued—short-time and long-time.

Method A—For each quadrangle the years of record for the several stations were summed; also, the total number of occurrences of a given storm intensity was counted. Dividing the former number by the latter gave the years interval for that particular intensity of storm.

By carrying out this process for a one-day storm intensity equal to 10% of the annual rainfall, and for successive $\frac{1}{2}$ -in. additions to this amount, a one-day frequency curve for the quadrangle was constructed. Similarly, a two-day curve was drawn, starting with a fall of 15% of the annual in two days. Three- and four-day curves were drawn where possible.

Fig. 6 reproduces the three-day frequency curve for the Miami quadrangle (9 E). It is evident that the desired maximum storm value is the greatest ordinate of this curve extended far to the right; and the maximum based on all experience in the same climatological region, not merely the Miami experience, would be found by getting the extreme ordinate of a similar curve drawn from the aggregate record of all the quadrangles.

Verification of the general shape of the curves is afforded by Fig. 7, a set of frequency curves computed for selected stations along the belt extending from Texas northeast to Maine, frequented by low-barometer areas. These curves, with others computed for the fortieth-parallel belt and for a north-south Mississippi Valley belt, reveal the existence of several distinct types of curve. Among these the Central States type is quite definite and flattens off early. This type is the significant one for floods and flood protection in the Miami Valley.

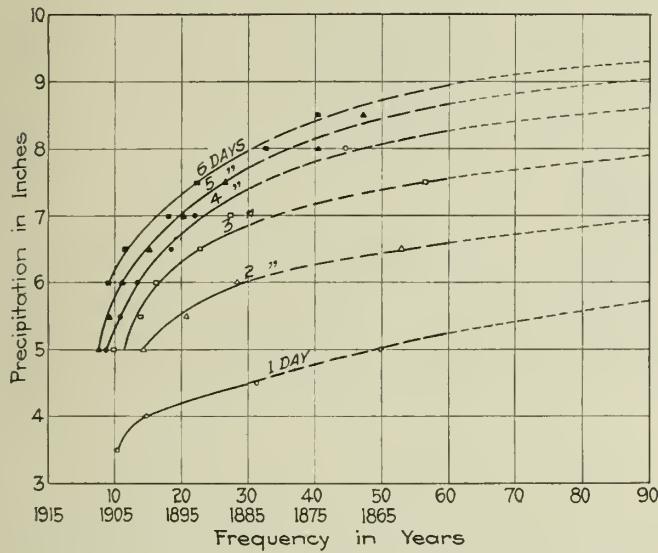


FIG. 6. QUADRANGLE FREQUENCY CURVES; THREE-DAY CURVES FOR MIAMI QUADRANGLE

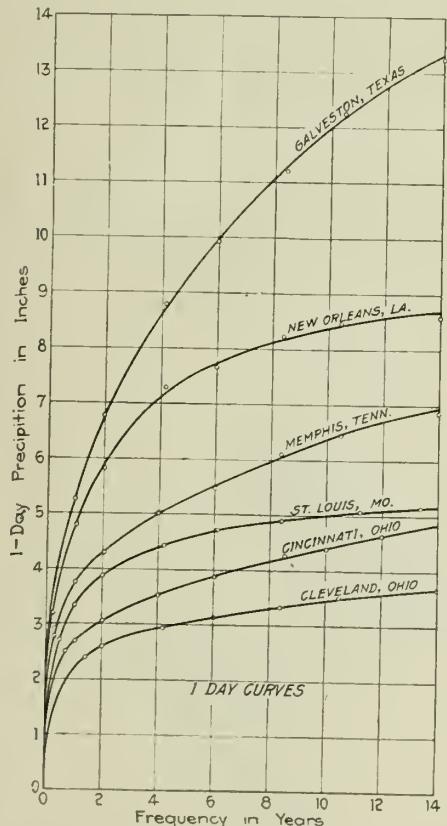


FIG. 7. SHORT-TIME FREQUENCY CURVES FOR ONE-DAY STORMS AT SIX STATIONS IN LOW-TRACK BELT

The method used for these curves gives slightly higher values than that used for Fig. 8, the latter being more correct. The 50-year one-day storm for Cincinnati totals $5\frac{1}{2}$ in.

Long-Time Storms; Method B—It was desired to compute the intensity of the storm that would occur only once in 50 years and once in 100 years, on the average. For this purpose the sum of the record-years of all the stations in a quadrangle, as 360, was divided by 50. The quotient (7.2) represented a number of storms, say seven. Selecting the seven largest storms in the record of that quadrangle and averaging their intensities gave the average intensity of the 50-year storm. Similarly, averaging the four largest storms gave the intensity of the 100-year storm.

This was done for all the 133 quadrangles. The only limitation was that the record at a single station must be at least 10 years.

Fig. 8 gives the 100-year three-day storm-intensity map of the eastern United States, whose data were computed by the process described. These values are close approximations to the ultimate values, for storms of three-day concentration period. Comparison of 100-year and 50-year storms shows that the maximum is closely approached.

The average ratio of increase of the 100-year storm over the 50-year storm, as shown by these figures, is 12 to 15%. This means that the 100-year value is not far below the maximum. Projecting the 50- and 100-year values forward into the future along the curve of the type shown by Fig. 6, the most liberal allowance will give an ultimate maximum storm value about 10 or 15% in excess of the 100-year value.

ANOTHER WAY OF SUMMING THE EXPERIENCE OF THE CENTRAL BASIN

A further development of the experience summation led to results that bring together the entire experience of the Central States. In this way, storm-incidence for very long periods of time could be studied, in spite of the fact that the records were for a short time only.

The area covered by this computation includes practically all of Illinois, Indiana, Kentucky and Ohio, and parts of Missouri, Iowa, Wisconsin, Michigan, West Virginia and Virginia—being the rectangle lying between 37° and 43° latitude and 81° and 91° longitude. There were records from 478 stations in this area, with record lengths ranging from 10 years to 73 years and averaging 15.8 years.

Fig. 9 shows the curves derived from these records. They indicate a slowly decreasing rise to a maximum, but a much slower rise than in Fig. 6 or 7.

The conclusions drawn from the curves were primarily applicable to areas as large as the entire Miami basin. To get values for the separate tributaries (with reference to the individual retarding basins), it was not thought desirable to attempt further mathematical deduction. The developed runoff curve (heavy full line in diagram at left (Fig. 2) was taken as applicable to the two large basins (Huffman and Taylorsville); a slightly higher curve (dash line) was drawn for the three smaller basins.

THE RAINFALL-RUNOFF RELATION

The studies as described were studies of rainfall. But runoff is less than rainfall, in variable ratio. Does present or future variation of its ratio affect the Miami flood problem? Can higher runoff occur, so that a given rainfall will produce a greater flood?

The 1913 crest flow is estimated at 84% of the maximum rainfall. The drainage area is so widely agricultur-

tural that even with bare ground and long rainfall a 100% rate was not reached. Future increase of runoff ratio by improved water-shedding property of the area was considered.

Most of the area is cleared, and its low portions are fully ditched. Tiling to replace ditch drainage is just beginning. Ditches give rapid surface runoff, but leave

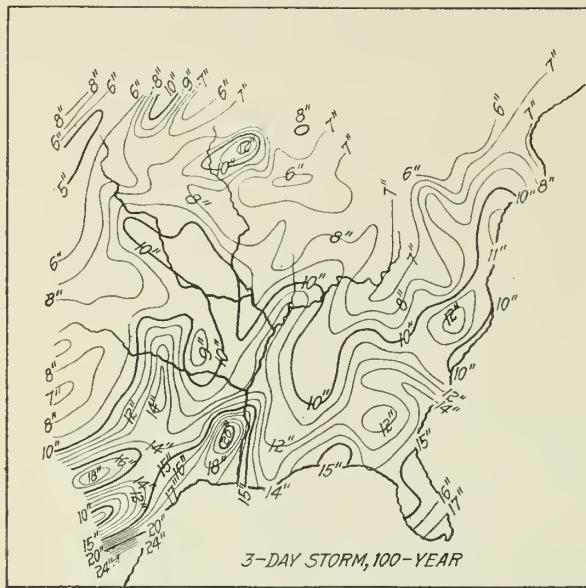


FIG. 8. THREE-DAY 100-YEAR STORM MAP OF THE EASTERN UNITED STATES

The contours and figures represent the total rainfall of the three-day storm whose average period of recurrence is 100 years

the ground saturated. Tile drainage carries the water away much more slowly, but continues the process until the excess of water is removed from a 2- or 3-ft. depth of soil, leaving a large temporary storage capacity. These reasons make it probable that extension of tiling will decrease the runoff ratio rather than increase it. The conclusion is confirmed by measurements of runoff from pumping drainage districts in Illinois.

THE SEASONAL INFLUENCE AND SNOW

While it was concluded that the runoff ratio cannot change for the worse, the chance that a great storm may find frozen ground was taken into account. In connection therewith, seasonal differences were studied attentively. Both rainfall and runoff vary with the seasons, but in inverse sense. The conclusion from a consideration of all the factors was that, with respect to long-time possibilities, the seasonal variations are largely compensatory.

Against the fact that in winter and spring the runoff ratio is greatest—with saturated ground and full discharge from the groundwater reservoir—there is the striking fact that winter storms are not only rare, but also less intense than summer storms. The curves of Fig. 5 show only the 1913 storm as coming in months of high runoff ratio. All the great storms in the area studied include only one other really great winter storm (Dec., 1895, Missouri, 8.78 in. in three days), but this occurred in a region of higher normal rainfall than Ohio.

The judgment of the engineers was that the differences between winter and summer storms and the possibility of frozen ground in late winter make it most reasonable to assume a runoff ratio about the same as that of 1913—

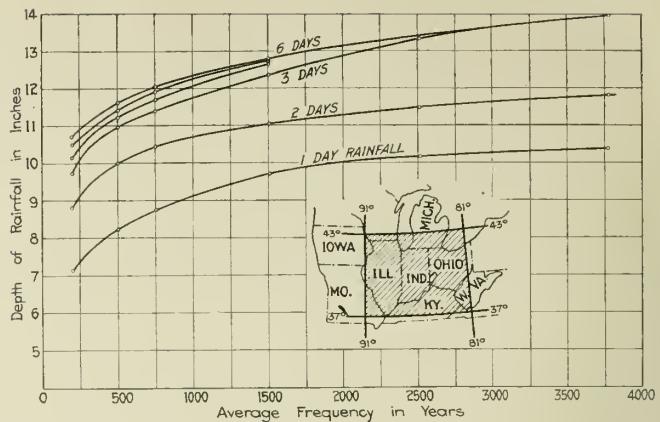


FIG. 9. AGGREGATE FREQUENCY CURVES FOR CENTRAL STATES BASIN

From records of 478 stations having an average period of record of 15.8 years. Curves show frequency of maximum rainfall

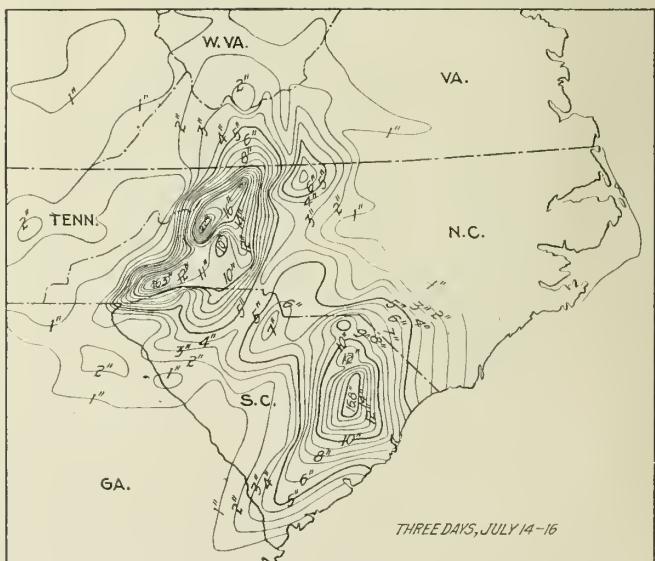
pamely, 85 to 90%—and therefore to base all assumptions of flood flow directly on comparison with the 1913 flood.

Snow as an additive element was—after full consideration—left out of account. There is no record of a great storm that fell on snow-covered ground. The nature of the air movements that produce long-continued storms is incompatible with the presence of snow or ice, it is held.

STORMS IN SEQUENCE, AND SOUTHERN STORMS

The soil storage in the Miami area during the summer season was very large. Careful experiments showed that it amounts to $2\frac{1}{2}$ -in. water depth. This factor constitutes a protection not only against the intense summer storms, but also against the distant chance that two great storms might follow in close sequence—a chance existing only under summer conditions. The soil drains rapidly enough so that in summer a 10-day interval between storms would suffice to prevent any large influence of the first storm upon the runoff of the second.

The tremendous semitropical rainfalls of the Southern States might suggest startling possibilities for the Miami Valley. The recent (July, 1916) Carolinas storm presents a serious picture, as see the three-day rainfall curves in Fig. 10 (see also the maps of progress of this storm



printed in *Engineering News*, Nov. 9, 1916, p. 887). But the fact is that such storms cannot cross the mountain barriers; their main burden of cloudwater must discharge as they travel up the slope. The Carolinas storm, for example, showed a tremendous total—18 in. of rain in two days on a 500-mi. area—in North Carolina; yet on the landward side of the mountain ridge, but a few miles west, in eastern Tennessee, only 1 in. of rain fell during the same period.

The storm studies previously described comprise the operation of all causes, including those which can bring Gulf or West India storms up to the Miami Valley. The question of how much water such a storm can deposit near

its place of origin, or in the coast states, is not pertinent. The important matter is how much it can bring to Illinois or to Ohio. These studies have already given the answer: Southwestern Ohio can never experience a storm runoff in three days greater than $9\frac{1}{2}$ or 10 in., for an area equal to the Miami Valley.

The studies of rainfall and runoff have been carried out under the personal supervision of Arthur E. Morgan, chief engineer, and S. M. Woodward, consulting engineer; G. H. Matthes has taken charge of working up the rainfall data, and Ivan E. Houk has had charge of the runoff and rainfall measurements in the Miami Valley.

The Miami Valley Flood-Protection Work

II—*Why Retarding Basins Were Adopted*

Development of Miami Protection Plans; Retarding Basins the Only Solution; How Basin and Channel Work Was Balanced; A Statement of the Factors That Favor Retarding Basins in the Miami Valley

The nine judges of the Miami Conservancy Court declared unanimously, only a few weeks ago, that retarding basins are the best and only practicable method of flood protection for the Miami Valley. Their finding was a re-statement of what the engineers had discovered nearly two years before.

The early investigation made by the Morgan Engineering Co. for the Dayton Flood Protection Committee, and later for a similar Miami Valley committee representing all parts of the river basin, was begun frankly on a channel-improvement basis. For the first month or two the problem assigned was primarily to find a means of protecting Dayton and incidentally to learn whether the problem was a local one or must concern the entire valley. Very soon the whole valley was made the primary ob-

quired to confine the waters. The railway entrance to the city would be blocked, and the city would be confined within ramparts, throttling its development. The west-side bypass channel required excavation and the placing of concrete 30 ft. below groundwater for miles, through a densely settled district, and at best proportions it entailed water velocities up to 20 ft. per sec.

These two cases picture the general unpromising aspect of all the diversion schemes. Channel improvement showed up little better.

The five concrete bridges crossing the river within the city were a first obstacle. It would have cost \$1,000,000 to raise them for a higher flood level, and a large additional amount to carry the piers down lower, for channel deepening. Probably tearing out the old bridges and

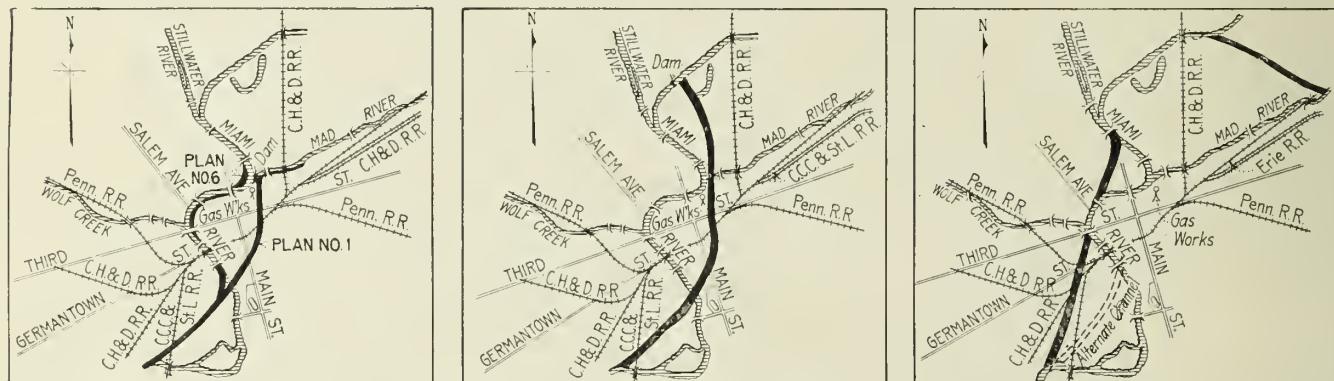


FIG. 1. SKETCHES OF SOME EARLY FLOOD PROTECTION SCHEMES FOR DAYTON

One-channel improvement and three diversion plans: Plan 1, at left, diversion of the Mad River by a dam at its mouth through a channel along the Miami and Erie Canal, in conjunction with two storage reservoirs on the Stillwater; Plan 2, in middle, has the same reservoirs, but a joint diversion of both Miami and Mad Rivers along the canal location; Plan 3, at right, diversion of Mad River into the Miami, with a west-side cutoff to carry the three rivers; Plan 6, at left, channel improvement through the main part of the city

jective, as it became recognized that success could come only through coöperation.

Construction of bypasses or cutoffs, widening and deepening of the river, levee raising, diversion of one or another branch of the river into a neighboring drainage area—these and all other possible schemes for dealing with Miami floods were studied. Definite cost estimates were made for most of them. The sketch plans grouped in Fig. 1 outline several of the preliminary schemes for the Dayton protection. Similar ones, equally radical, were devised for a few other parts of the valley.

FULL TRIAL OF BYPASS PROJECTS

Two among the projected methods for protecting the City of Dayton are characteristic: The first had as its main feature a huge reinforced-concrete culvert through the city along the line of the old Miami & Erie Canal, on the east bank; the second involved open bypasses on the west bank. It was found possible to put the east-side box culvert through the city at a cost of perhaps \$15,000,000. The city would be cut apart by a 10-ft. wall. Auxiliary levees above and below the city would be re-

building new ones would have been the best course. However, this was the least of the difficulties encountered.

By widening the river through high-priced property, as much as possible; deepening its channel 10 ft.; paving the bottom to withstand high velocities; and, in some of the variations, raising the existing levees by amounts of 3 to 5 ft., the 1913 flood flow could be carried through the city. How the water could be cared for at the sloping drop (or dam) required at the upper end of the city to deliver the flow safely to the improved channel remained a problem.

At the most moderate estimate the cost of the whole work would run up to \$16,000,000 or \$20,000,000. This would protect Dayton alone. A large annual maintenance expense had to be faced in addition. A further complication of the case appeared at this time. Computations of the amount of water stored in the overflow portions of the valley during the 1913 flood showed that this storage was of enormous amount. With full-capacity channels, giving a more rapid flood wave by eliminating the valley storage, the greatest flow at Dayton would be raised from 250,000 to 300,000 sec.-ft., and at Hamilton from 350,000

to 500,000 sec.-ft. The increased figures made the problem virtually impossible.

From the first, attention had been given simultaneously to all possible means of protection. The survey parties reconnoitered for reservoir basins and dam sites, while they were taking flood levels, topography, river slopes and geological memoranda. Elaborate studies of what reservoir control has accomplished in other parts of the world were started in the summer of 1913. Control by "dry reservoirs"—retarding basins—was suggested at the same time. Data were being accumulated that placed increasing emphasis on storage or detention possibilities.

INVESTIGATION OF BASINS FOR FLOOD REDUCTION

The ultimate result of the work, by the end of 1913, was that local protection—by channel improvement, by bypasses, by levees—had demonstrated its impracticability. Basin-storage projects were now outlined and studied, though at first with not much faith. Very shortly the planning was centered on dry basins, as giving obviously the maximum flood reduction. Further steps in the progress of thought soon introduced the idea of minimum-time detention—that is, making the permanent outlets of the basins just as large as the channels below could be adapted to. The three advantages secured by quick emptying of the basins were (1) that the farm lands in the basins would remain wet the shortest time; (2) that the basins would quickly be ready for new service in the improbable case of a second storm coming soon after; and (3) that during the early part of the storm the capacity of the basin was being held empty and available for storing the crest of the flood.

The unpromising flat valley of the Miami and its tributaries was found to possess excellent basin locations, with dam sites permitting the construction of moderate-cost dams and affording enormous volumes of storage. These sites were studied in detail. Many reservoirs were laid out, their influence on flood flow was studied, and various combinations were made.

It was then found that the available reservoir storage was large enough, and sufficiently well distributed, to enable complete flood control to be obtained for the valley as a whole. Several possible combinations of reservoirs, supplemented by moderate channel improvement to carry their discharge, were found to cost less than the Dayton local protection by channel enlargement.

This discovery focused all further work on control by retarding basins. The best basin combination was worked out, and trial estimates of various amounts of channel improvement were made, to balance against the cost of increased reservoir capacity, in order to arrive at the final project. Results in substantially their present form were reached by the beginning of 1915.

In the meantime the state legislature had passed the act allowing conservancy districts to be formed, so that the machinery for joint action of the valley was available. The clinching argument for joint action lay in the fact, shown by the full estimate of cost, that no part of the valley could provide for itself alone at less cost than its share of cost of the general project, or at as low a cost.

Five retarding basins were embraced in the scheme, as the map, Fig. 2, exhibits. Three lie just above Dayton, in the three main streams that join within the city—the Taylorsville basin on the Miami, the Huffman basin on the Mad River and the Englewood basin on the Stillwater.

TABLE 1. DATA OF THE RETARDING BASINS

Basins	Dam Height	Drainage Area, Ft.	Spillway, at Spillway Elevation	Water Surface Acres	Acre-Feet	Three-Day Instantaneous Outlets		Capacity at Spillway Inches of Runoff on Drainage Area
						Capacity	Time of Filling with Discharging	
Germantown.....	107	1,210	270	815	3,520	106,000	7.30	10.75
Englewood.....	124 $\frac{1}{2}$	4,660	651	876	7,930	312,000	8.99	10.70
Lockington.....	78	6,400	255	938	4,020	70,000	6.68	10.00
Taylorsville.....	78	2,980	1,133	818	11,000	186,000	4.42	9.51
Huffman.....	73	3,340	671	835	9,180	167,000	4.66	9.52
								35,650 841,000

These throttle the extreme concentration that takes place here at the base of the main fan of the valley. Of the other two, the Lockington reservoir controls the upper Miami and thereby protects the cities of Piqua and Troy, while the Germantown basin, on Twin Creek, reduces an important flood contribution to the Middletown flow.

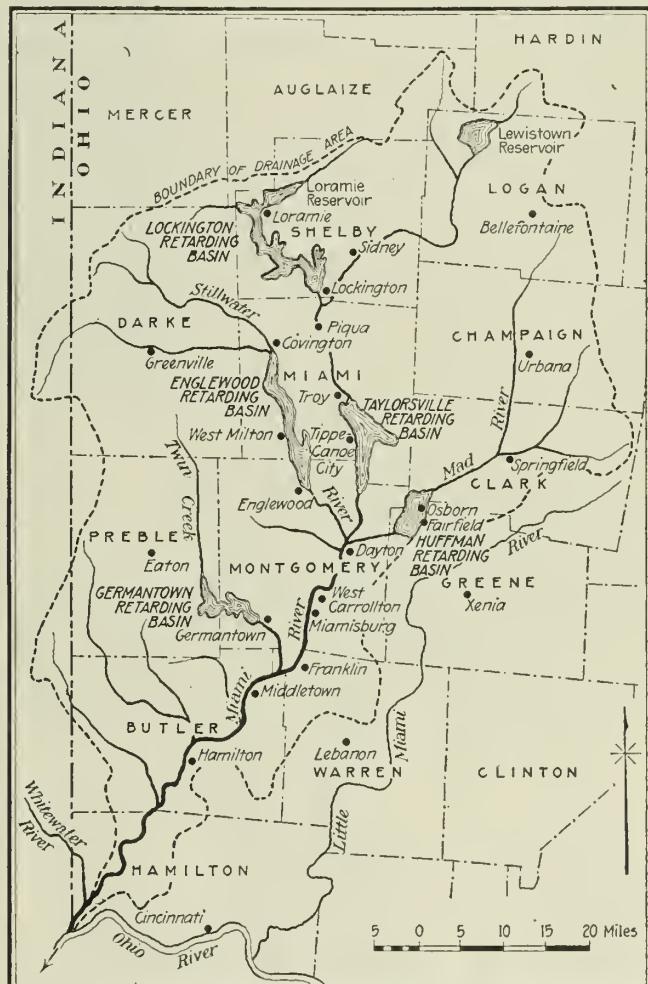


FIG. 2. MAP OF MIAMI VALLEY, WITH LOCATIONS OF THE FIVE RETARDING BASINS TO BE BUILT

Table 1 gives numerical data on the basins and their dams. Each dam is pierced by two to four large conduits, permanently open, located at the base of the dam. These are to pass the normal river flow, and in floods are to discharge water under head. No gates, stoplog notches or other closure devices are fitted. The entrance is protected from drift by piers and a floating boom.

Enormous volumes of water will be discharged from the conduits at maximum flood. It should be remarked that the conduits are so proportioned in relation to drainage area and basin capacity as to let the basins fill approximately to spillway level in the assumed ultimate maximum flood. Spillways are provided, with 15-ft. to 19-ft. freeboard below dam crest, as a pure safety margin, which

amounts to about 40% over the maximum flood. In case of a flood twice as great as that of 1913 the dams would have a freeboard of 5 ft. The insurance of this freeboard in such a case finally determined the heights of the dams. The Germantown, Englewood and Lockington dams required a greater freeboard than 15 ft. above the spillway to meet the second condition.

Table 2 gives the conduit discharges in second-feet for the ultimate maximum storm, when the basins are filled to spillway level. These discharges involve velocities of flow amounting to over 55 ft. per sec. at several of the dams.

TABLE 2. CONDUIT DISCHARGES FROM THE BASINS WHEN FULL TO THE SPILLWAYS

Basin	Discharge Sec.-Ft.
Germantown	10,000
Englewood	12,000
Lockington	8,830
Taylorsville	53,600
Huffman	35,000

BALANCING BASIN AND CHANNEL WORK

The most desirable combination of basins and channel improvement would be that one costing the least to build. This balance could be attained in the Franklin County Conservancy project (Columbus, Ohio). In the Miami project it is nearly attained, but not quite. The limiting factors were certain conditions affecting the basin storage.

To develop the ideal capacity of the Taylorsville basin would submerge much of Tippecanoe City, a village some five miles above the dam. Troy, lying several miles farther north, would possibly also be affected. Similar conditions

apply to the Huffman basin, where the cost of more radical railway readjustments would be prohibitive.

In consequence these two basins are of smaller capacity than the others, relative to their position and drainage area. The result is seen (Table 2) in the large discharge rates of 55,000 and 35,000 sec.-ft. for the two basins when full to spillway. The sum of these two figures alone gives a flood flow equal to the entire present capacity of the Miami River Channel at Dayton.

With respect to the other basins, it may be said generally that larger capacity could have been secured without difficulty. The additional benefit would be small, however. The Englewood basin already has a capacity so large that three weeks will be required to empty it after the maximum storm. This basin, by the way, supplements Huffman and Taylorsville in vital manner, because it reduces the Stillwater's contribution to flow at Dayton from 88,000 to 12,000 sec.-ft. The Germantown basin will not economically bear much enlargement, because the flow is already reduced from 66,000 sec.-ft., in the case of the 1913 flood, to 9000 sec.-ft.

In spite of these modifying factors it may be put as an approximate fact that just so much channel work is to be done and basins of such size provided, that the total cost of the whole enterprise is a minimum, while securing full protection to the entire valley in all great storms.

The cost figures are briefly summarized in Table 3. Of the figure for channel work and local protection, only a little over \$2,000,000 represents cost of channel enlargement, or work that might be saved by increasing

TABLE 3. COST OF MAIN ITEMS OF THE MIAMI WORK

Retarding basins, not including real estate.....	\$6,735,000
Real estate in retarding basins.....	3,500,000
Channel work and local protection.....	3,468,000
Real estate other than in basins.....	2,203,000
Public utilities, relocations and damages.....	2,307,000
Total, not including interest, taxes, administration and contingencies.....	\$18,210,000

the storage. Against this stands the figure of \$5,000,000 as the cost of increasing the storage by any material amount.

FACTORS FAVORING RETARDING BASINS IN THE MIAMI VALLEY

The great novelty of the Miami project invites attention to the special features contributing toward the easy superiority of retarding-basin control in this valley. An inquiry indicates the following points as bearing on the question:

The River Is Flashy—A remarkably high ratio of maximum to minimum flow is found in the Miami, about 1000. Floods are not only large, but sudden. The river is characterized by two-day to three-day floods, with flows of 100 sec.-ft. per sq.mi. The Seine at Paris, which is not without serious floods, carries at the maximum 87,000 sec.-ft., from a drainage area of 17,000 sq.mi.; its floods last a month or two. The Miami, at Dayton, with one-seventh as large a drainage area, has floods three times as great, which are over in four or five days. The valley has a high concentration rate, due to its highly fan-shaped arrangement and to its straight supply channels.

Retarding basins are concerned primarily with the aggregate discharge: hence slow, long-continued floods preclude their use. Channel improvements have to deal with *maximum rate* of discharge, irrespective of aggregate discharge; hence channel improvement is the more difficult the more flashy the stream. The latter consideration

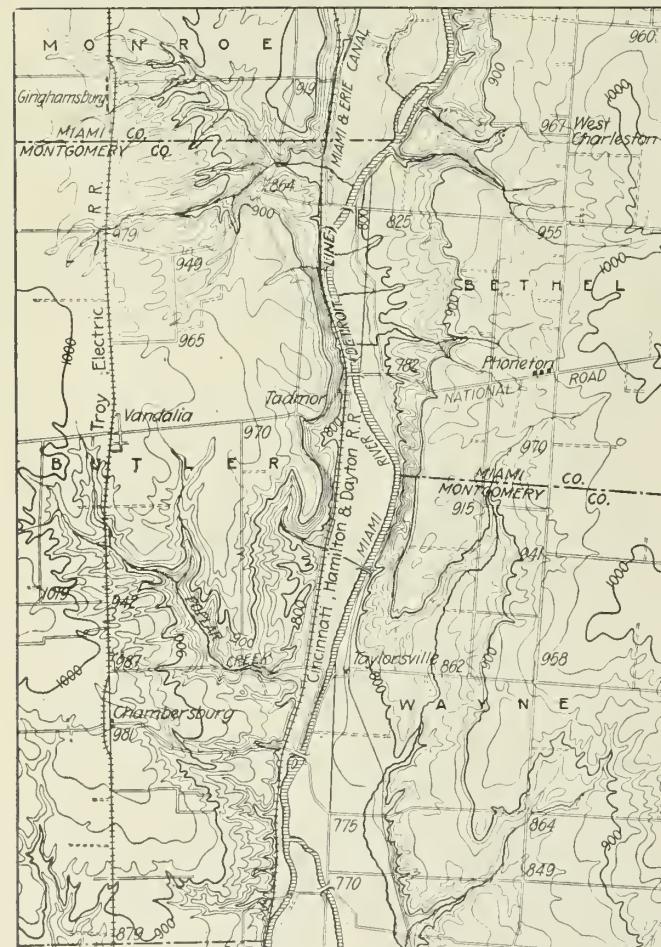


FIG. 3. CHARACTERISTIC TOPOGRAPHY OF MIAMI VALLEY, AS SHOWN BY MAP OF TAYLORSVILLE REGION

applies with special force when the greatest floods are to be provided for, as was desired in the Miami.

Good Basins—The region is flat, and to the eye appears devoid of reservoir possibilities. Yet its valleys are wide but well-defined steep-sided grooves in the upland. They have a flat longitudinal slope and a level floor transversely. Thus they are capable of providing very large storage volumes, if a dam site can be found. The topography is represented characteristically by the map, Fig. 3, at the Taylorsville dam.

Ample Storage Capacity—The storage capacity that can be developed is not so closely limited as to be adapted

or towns, the development of adequate basins would involve considerably greater cost in the way of damages.

Many Dam Sites Available—Good dam sites exist (see Fig. 4); in fact, there are more than need be utilized. Due to the shape of the valley sections, moreover, the dams are cheap. Those actually designed cost \$6 to \$16 per acre-foot of storage capacity, or about \$300 per million cubic feet on the average (\$13 per acre-foot).

The Miami plan is unique in being a device for securing *complete* protection against the largest floods ever possible. In this respect as well as in the admirable way

it was worked out, it stands as a model,

and as an object for most careful study

by future workers in flood protection.

The fact that the valley is rather un-

usually well adapted to basin control is

perhaps the chief or only natural con-

dition that facilitated the engineers'

planning. An ample supply of money

for full investigation was the great out-

side advantage. Arthur E. Morgan has

been Chief Engineer of the project

from its initiation; Daniel W. Mead, a

member of the Board of Consulting

Engineers, acted as Chief Engineer of

the District during an illness of Mr.

Morgan, and was frequently consulted

during the preparation of the general

plan, the working out of the details of

construction, and the writing of the

specifications; S. M. Woodward, Con-

sulting Engineer, and Kenneth C.

Grant were responsible for the hydraulic



FIG. 4. GERMANTOWN DAM SITE, SHOWING VALLEY TOPOGRAPHY IN THE MIAMI AREA

The other dams are much longer, but abut against valley sides of similar form and character

only by lucky chance to the ultimate maximum flood flow. Arthur E. Morgan, Chief Engineer, expresses the view that if the flood runoff to be protected against were as great as 20 in., in place of 10 in., yet retarding-basin control would still be the principal hope (or better, the only hope).

It is important to observe that villages are rather thinly scattered in the valley. Were there more frequent villages

studies and calculations throughout the preparation of the plans; O. N. Floyd, engaged upon the project from its beginning, personally directed much of the field work, has kept in close touch with all the engineering details, and has been largely charged with the innumerable negotiations with public service companies during the development of the project. C. A. Bock has been Office Engineer during the preparation of the plans.

The Miami Valley Flood-Protection Work

III—Dam and Outlet Problems

Proportioning Large Earth Dams and Their Spillways; Design of the Great Outlet Conduits; Discharge Channels Designed to Destroy Energy of Discharge by Standing Wave; Details of Design Developed by Experiments

Earth dams are to be used for protecting the Miami Valley against future floods. Because these dams must be safe under all conditions, a century from now as well as when just completed, their design was studied with unusual care. The engineers aimed at ample safety of the structures, and definite knowledge with respect to all conditions of service and operation. This is best shown by the adopted dam section, Fig. 2, and by the elaborate course of large-scale experimenting through which the discharge channels were developed.

These dams will be located above large cities, and their failure would result in enormous damage. They will stand dry for years at a time and then may suddenly be

Protection of the conduit entrances from obstruction by drift carried down in floods also called for attention. The engineers did not look on drift as a serious source of trouble, but the fact that residents of the valley had strong fears in this regard made it desirable to provide efficient protection.

DESIGN OF THE DAMS

Satisfactory dam sites were located for the construction of earth dams. Thorough exploration by borings was undertaken in order to find rock foundations for the outlets and spillways. This search succeeded, although one of the dams had to be shifted from the preliminary

location to get the desired rock foundation for the spillway at suitable depth. The rock foundation at all the dam sites, except Lockington, is geologically known as the Cincinnati formation and consists of layers of hard limestone interstratified with shale or hard clay. Investigations have satisfied the engineers that this formation will provide a foundation well suited for the outlet works and spillways. At the Lockington dam site the ledge rock is Cedarville limestone, a comparatively hard rock of massive bedding. At those dams which have spillways outside the dam structure itself, satisfactory rock channels for the spillways were found. The dams themselves in all five cases will be built on earth bottom. This condition and the nature of the materials available near-by dictated earth-em-

bankment construction. Basin capacities were fixed by the spillway levels chosen. Generally, these were governed by natural limits of elevation—the adjoining ridge elevations or the location of towns that could not be flooded (as at Taylorsville) or the limits imposed by economy in railway readjustments (as at Huffman). At Englewood the economical and desirable capacity of the reservoir, rather than the height of dam, was the limiting consideration.

Since a large spillway-flow depth was desired—for safety margin above all flood forecasts—the heights above spillway crest were worked out on a uniform basis. To this end an imaginary flood runoff of 14 in. in three days, which is no less than twice as large as that of 1913, was assumed; with spillway lengths determined by other considerations, the overflow depth was computed (the conduit discharge continuing at the same time). This gave depths ranging from 10 ft. at some of the dams to 14 ft. at others. A uniform freeboard of 5 ft. above this imaginary water level was then adopted, which fixed the dam-crest elevations.



FIG. 1. EXPERIMENT STATION WHERE HYDRAULIC-JUMP TESTS WERE CARRIED OUT

subjected to full head of water, a condition which in itself raised question on the part of some residents of the valley. They must not only be safe under normal conditions, but also safe even if supervision and maintenance should become lax. They must be foolproof. Their outlet conduits must be permanently open and unblockable. The discharge of large volumes of water at high spouting velocities must be rendered powerless to do damage.

The greatest of these problems, perhaps, was that of the discharge conduits, which under maximum conditions will deliver the water at a flow-velocity of nearly 60 ft. per sec. The Taylorsville dam, which has the largest conduits, is designed for a discharge of 55,000 cu.ft. per sec., nearly as much as the present safe flood-flow capacity of the 600-ft. channel of the Great Miami at Dayton. The power residing in this discharge must largely be dissipated before the water can safely be delivered to the river channel below. Even within the conduits the destructive ability of the high-velocity flow is so great that local defects in the structure of the conduits might lead to destruction.

With respect to the proportioning and methods of construction of the dams, the existence of some popular prejudices against dams—and especially against earth dams—was held to justify excessive safety provision.

The fears were concerned partly with the fact that the dams would be dry for years and then might suddenly be exposed to full service, with maximum head of water stored in the basins, for periods of one to three weeks (the Englewood basin, which has the highest dam and for its storage the smallest conduits, will take three weeks to empty after the assumed maximum flood). Although there was virtually no direct experience on this subject that could be quoted, the engineers were at all times convinced that the dry dams would be *far safer* than dams that are constantly wet and exposed to percolation under head.

The soil in the valley and on the adjacent upland is glacial till, comprising gravel, sand and clay in varying proportions. Prospecting and sampling proved that tight mixtures were either naturally available or could be put together. This fact was verified carefully, by analyses of grain size and by the expert judgment of men long engaged in earth-dam construction. It simplified the questions of foundation and of dam design quite materially.

NO CORE WALL OR LINING; CUTOFF TRENCH USED

Construction of an embankment by either roller compacting (in layers) or by hydraulic deposition was decided to meet all requirements, without lining or core wall. A cutoff trench to go down 30 ft. or so, well below the surface layers, will be used.

The section adopted, Fig. 2, is distinctly more ample than that of the latest and strongest existing dams on tight or semipermeable foundations—though, of course, not comparable with the Wachusett or Gatun type. It is proportioned for specially wide base. The features are

any other cause is held to be vanishingly small with gutters, as compared with buried pipes.

The cutoff trench is indicated in Fig. 2, although local conditions will determine its depth. It is intended mainly to give most intimate connection between the impervious dam core and the subsoil, and thereby prevent seepage along the base. In all cases the dams will be built on ground stripped of top soil. The subsoil contains very little bedded porous material, so far as the borings and test pits revealed; in the process of making wash borings, the pipe lost its water only rarely. Geological indications are that any porous deposits are local—that is, have little horizontal extent. It is also important to recall that underwashing of a dam is a slow process, while here

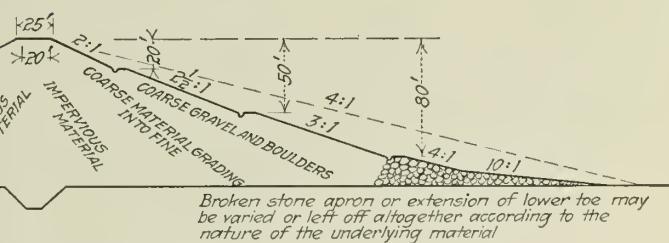
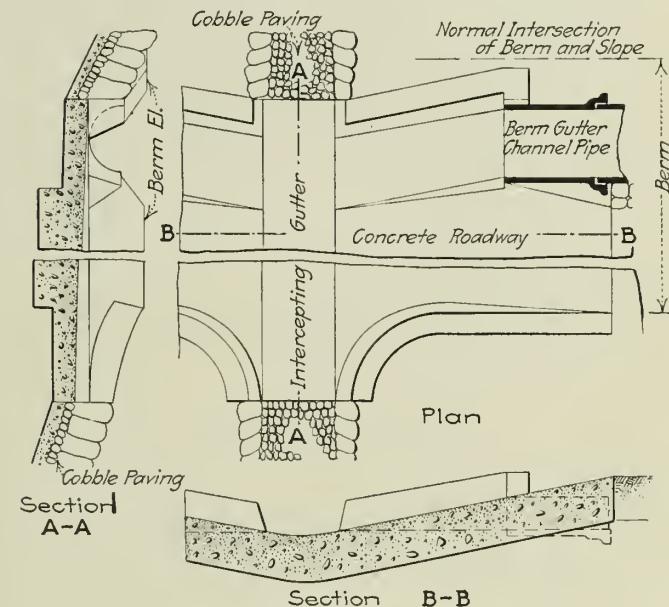


FIG. 2. TYPICAL CROSS-SECTION OF MIAMI CONSERVANCY DAMS

With detail of berm and gutter shown above

frequent berms, concaved sides and symmetrical outline—that is, upstream and downstream faces alike (because these are dry dams). Compared with the standard embankments of the Board of Water Supply of New York City, the upper berm is nearer the top and the slopes flatten out more toward the bottom, to a maximum of 4 to 1. Toe protection of broken stone sloped 10 to 1, as shown in the sketch, may be added if found convenient or desirable.

The slopes are to be grassed, top soil being placed on the structural body of the embankment for this purpose. It is intended that they shall be kept trimmed and neat at all times after construction, as part of the regular maintenance work of the district.

Slope drainage (for surface water) is accomplished by paved berm gutters and connecting gutters down the slopes. The chance of deterioration from settlement or

the water will never stand behind the dam more than a short time.

At the Huffman dam, in Mad River Valley, the most porous ground was encountered (Fig. 5). For purposes of calculation this material was assumed to be as open as very coarse sand. Yet the computed velocity of outflow proved to be almost infinitesimally small ("five times the velocity of the long hand of a watch"), although the extreme case was taken of the outflow concentrated in a 3-ft. width at the toe of the dam.

It is believed that the dam body as shown by the section, with the materials available in the valley, will be permanently sound, and proof against accidental or malicious injury as well as against deterioration.

The vital element of these dams lies in the outlet conduits. They must always be open, or the retarding basins lose some of their protective power. They must be struc-

turally sound, or the high discharge velocities might tear them to pieces and wash out the dam. Straight flow-lines and smooth, durable surfaces must be obtained. The Germantown and Englewood conduits must carry the weight of 120 ft. of earth above, without cracking or de-

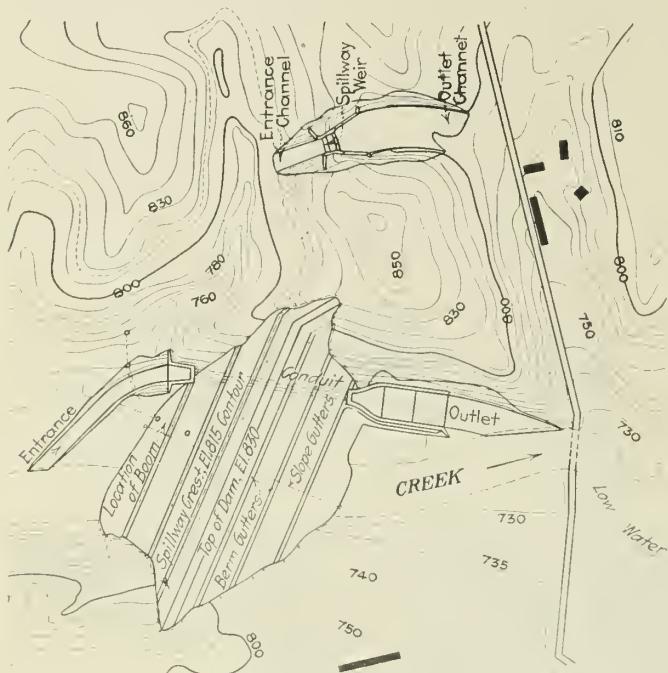


FIG. 3. PLAN OF GERMANTOWN DAM, SHOWING SEPARATE SPILLWAY

forming, even when slight settlement or shrinkage of the embankment takes place. They must be bonded tightly into the embankment, so that water cannot flow along the outside of the walls.

The prime importance of open outlets made it desirable to employ two (or more) conduits rather than one. If surface wear of the interior requires relining or the like, one conduit at a time can be taken in hand without endangering the operation of the system.

Separation of conduits and spillways (Fig. 3) was attempted in all cases, at first. The advantage of a spill-

way built in natural ground was considered decisive, while the conduits could not well be tunneled through the hillsides, but had to pass under the embankment. On further study, however, especially with regard to construction procedure, it was found that a combined outlet-and-spillway structure located within the embankment possesses marked superiority, at least for low dams. Such a structure solves the problem of stream control while the embankment is being carried up. For this reason the Taylorsville, Huffman and Lockington dams were provided with combined structures. At the two high dams, Germantown and Englewood, the separated spillway and conduits were found to be more practicable and economical.

The plans, Figs. 3 and 4, represent the two types—the separated structures of Germantown dam and the combined structure of Huffman dam. The difference of length of conduits should be noticed in Table 1, 725 ft. against 40 ft. Since the friction loss in the long conduits amounts to more than 10 ft. of head, the longer conduits must be appreciably larger—and therefore more costly—to secure the desired discharge capacity. Further, the long conduits involve more complication in the way of transverse joints, cutoffs, etc., and structurally present more difficult problems of design. On the other hand, with a combined structure the retaining walls that form the spillway abutments become enormously expensive when a height of 100 ft. is approached, as at Germantown and Englewood.

The proposed construction procedure is to build first the concrete trough formed by conduit floor and spillway abutments, leaving out the spillway and conduits. This trough forms an ample stream channel, through which the river is diverted. Construction of the dam then proceeds under safe conditions. Finally the concrete trough is bulkheaded off so that the conduit division walls can be concreted. As the last step, the spillway body is built, this work lying well above normal stream level. Deep notching of spillway into side walls will take care of downstream thrust of impounded water.

Cross-sections of the conduits are given in Figs. 6 and 7. For the combined structures, substantially rect-

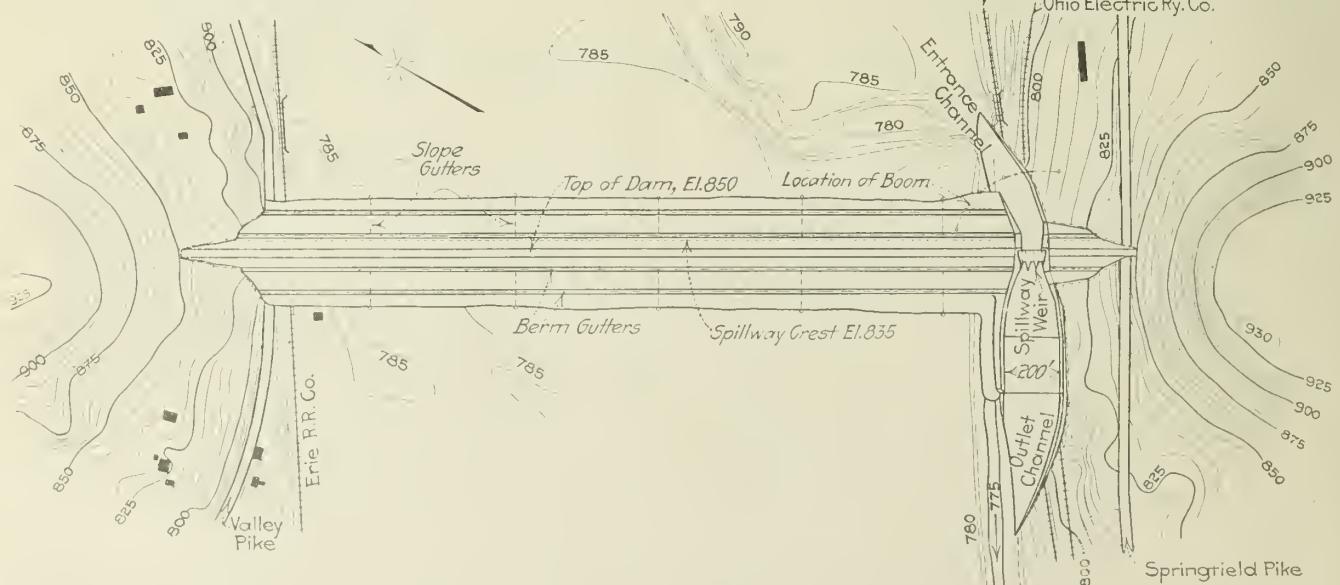


FIG. 4. PLAN OF HUFFMAN DAM, WITH COMBINED SPILLWAY AND CONDUITS

angular sections were adopted, while the separate conduits are of horseshoe shape. A rich lining is to be used—a $1:1\frac{1}{2}:2\frac{1}{2}$ concrete placed in the forms integral with the body—for resistance to abrasion. Whether hard-brick facing of the invert will be used is not decided. Table 1 shows the maximum conduit velocities in cases of great floods. These velocities are high, but not as high as in the case of several other structures where concrete conduit linings are giving perfect satisfaction. At Arrowrock dam, for instance, the concrete outlets have been subjected to velocities as high as 70 ft. per sec. for several weeks at a time during the last two years, and to velocities above 60 ft. per sec. for much longer periods, with no indication that the concrete lining will not remain in good condition indefinitely (Charles H. Paul). Many other instances are on record where carefully constructed concrete linings are withstanding velocities as high as 60 ft. per sec. with no signs of wear. And in practically all these cases the periods of high velocity are much more frequent and much longer than will be the case with these.

TABLE 1. DISCHARGE-CONDUIT VELOCITIES

	With Ultimate Maximum Flood, Ft. per Sec.	At 14-In. Flood, Water 5 Ft. Below Dam Crest, Ft. per Sec.		Length of Conduits, Ft.
		55	59	
Germantown	55	59	520	
Englewood	56	60	725	
Lockington	56	61	45	
Taylorsville	48	57	40	
Huffman	50	59	40	

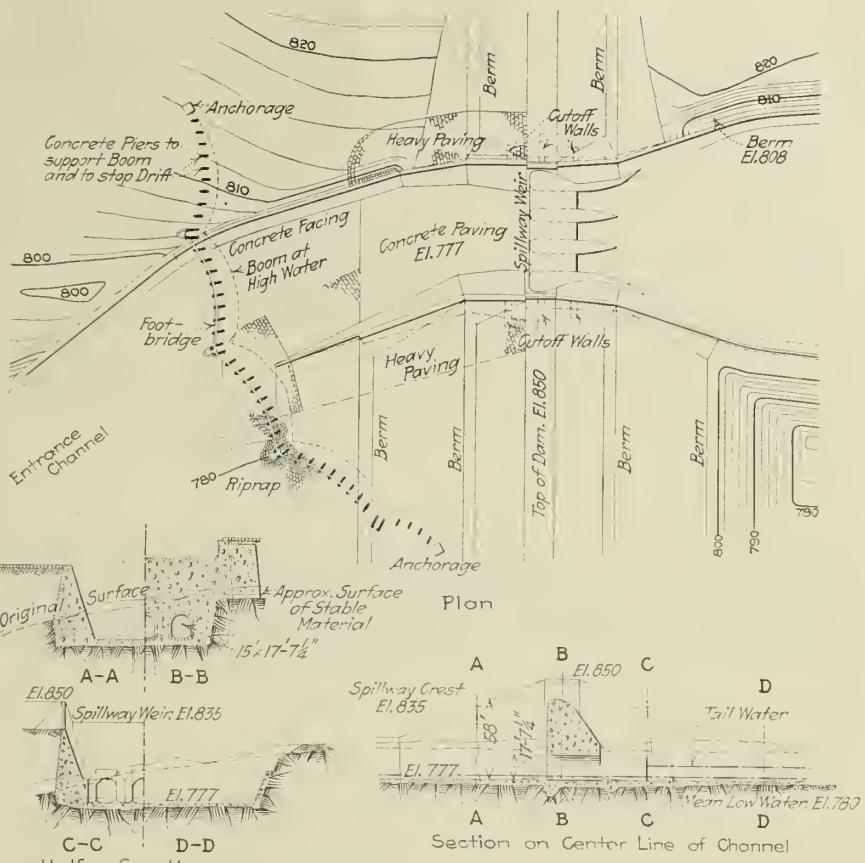


FIG. 6. COMBINED SPILLWAY-AND-OUTLET STRUCTURE OF HUFFMAN DAM

The entrance end of the conduits is flared or bellied out slightly, to avoid throttling under full flow. Above here the entrance channel is tapered smoothly, for good approach, and beyond the concrete approach platform the earth surfaces of channel and embankment are paved with

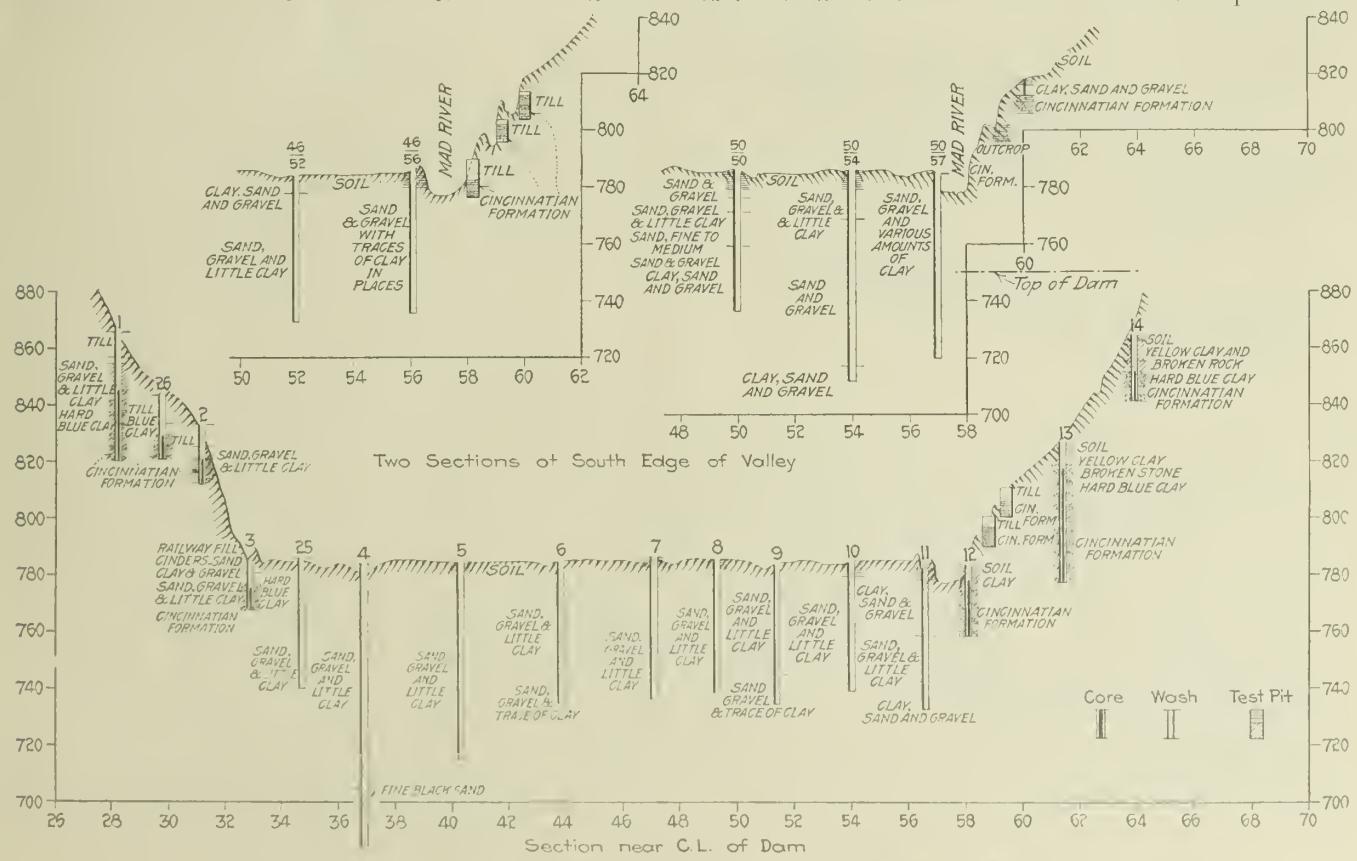


FIG. 5. SECTION OF MAD RIVER VALLEY AT HUFFMAN DAM SITE

stone to prevent erosion. At the lower end, the conduits open out on a discharge channel, a remarkable structure of critical importance, whose development by experiment is noted farther on. Belling the outlet ends of the conduits was considered, but rejected.

The long conduits presented various structural problems. They carry over 100 ft. of fill at midlength and not more than 10 or 15 ft. near the ends. In some cases the rock on which they will be founded may be capable of taking only comparatively low bearing pressures with safety; and although no settlement is anticipated, it was

(probably formed by glacial erosion) goes down sharply from the valley sides to great depths—well over 100 ft. at most of the sites (see Huffman site section, Fig. 5).

A factor in the structural design was the desire to avoid the use of steel reinforcement above low-water level. The conduit walls and arch were therefore designed of mass section. The floor was at first planned out as a plain arch invert. With footing pressures limited to 7 tons and a load of 64 tons per lin.ft. of wall (at Englewood), the floor must distribute the wall loads of a single conduit over 18 ft. of width. To do this in reliable manner by

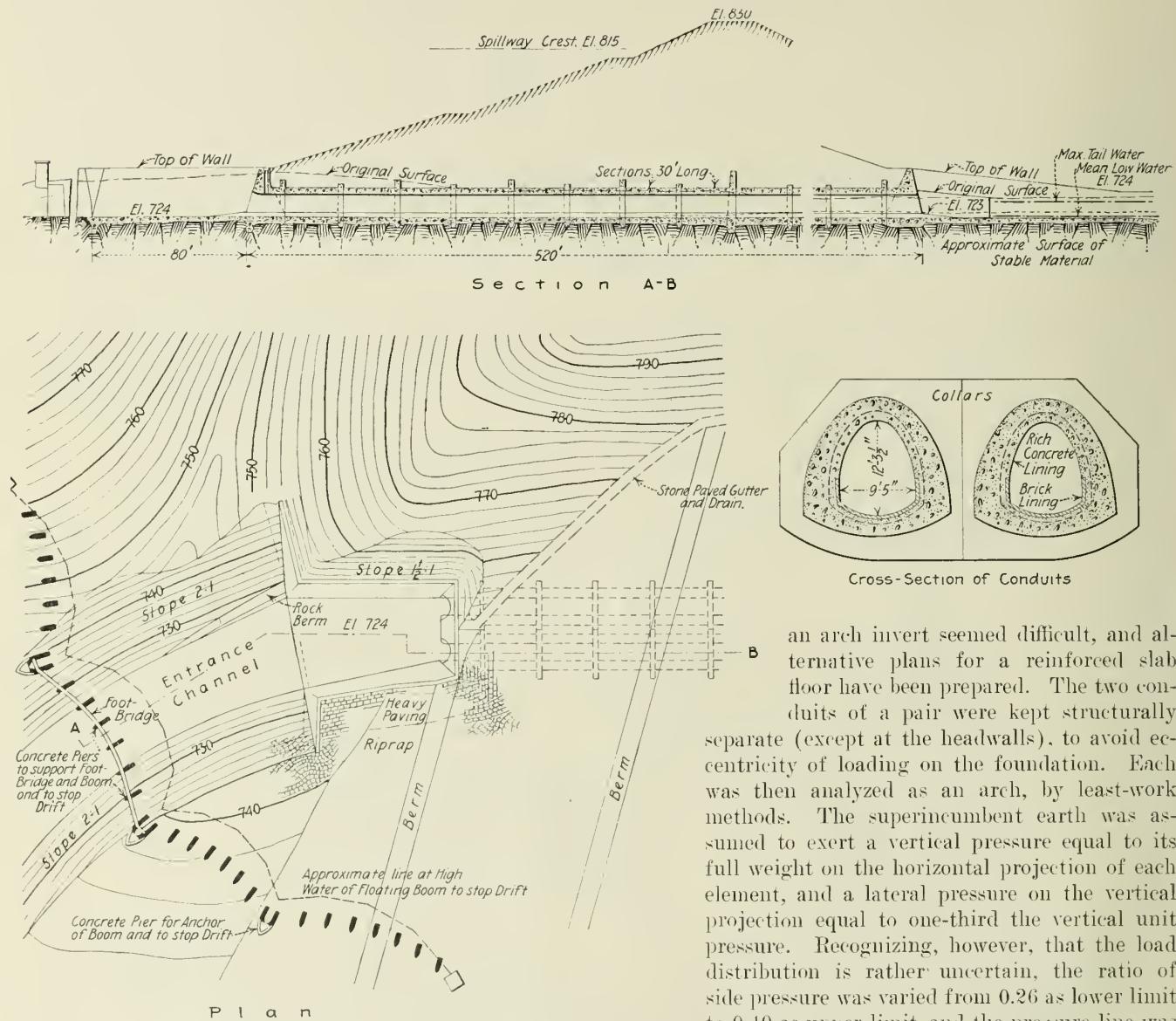


FIG. 7. GERMANTOWN OUTLET CONDUITS
Plan, profile and cross-section of conduits

thought advisable so to design the structure that it would accommodate itself to slight settlement, if necessary. The questions of stress distribution and arch action needed study, therefore.

The foundation rock, the Cincinnati formation, varies considerably in load capacity, depending on whether the clay layers are already consolidated to shale or remain of clay nature. In the latter case the allowed loading will be limited to what hard, confined clay can carry. There will be no serious difficulty in designing to this limitation. The strata all are horizontal, although the rock surface

an arch invert seemed difficult, and alternative plans for a reinforced slab floor have been prepared. The two conduits of a pair were kept structurally separate (except at the headwalls), to avoid eccentricity of loading on the foundation. Each was then analyzed as an arch, by least-work methods. The superincumbent earth was assumed to exert a vertical pressure equal to its full weight on the horizontal projection of each element, and a lateral pressure on the vertical projection equal to one-third the vertical unit pressure. Recognizing, however, that the load distribution is rather uncertain, the ratio of side pressure was varied from 0.26 as lower limit to 0.40 as upper limit, and the pressure line was kept in such a range of position that the unit stresses in the concrete would never exceed the allowed pressures by more than 20% due to change in ratio.

With regard to compressibility of the foundation strata and temperature stresses, the conduit arch is to be made virtually three-hinged, by placing construction joints at bottom of side walls and at crown. Interposition of a thin cushion layer—as asbestos felt—in these joints is expected to insure perfect adjustment of the arch to its load and to temperature variation. Without the three-hinged arrangement the computed temperature stresses were serious (up to 250 lb. per sq.in.).

The structural calculations were made for two thicknesses of wall—that is, with and without the lining layer. This will insure safe stability even during a period of repair, when, for example, the lining might be cut out for replacement.

Longitudinal shrinkage is taken care of by building the conduits in 30-ft. sections. These are to be laid up concrete against concrete and are not intended to give room for expansion, as it is expected that the work will

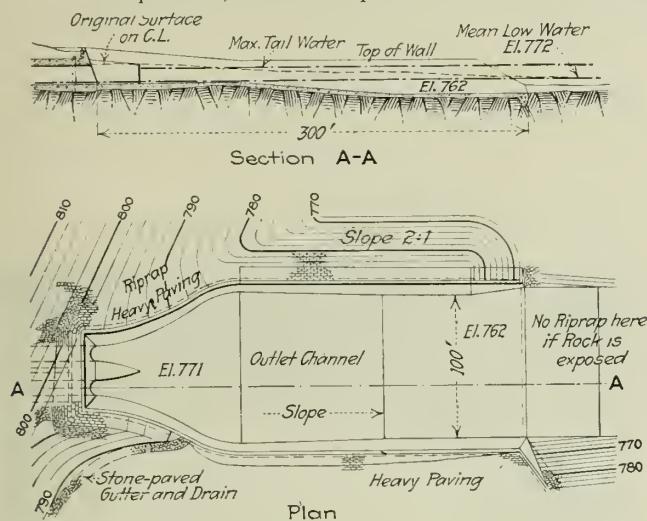


FIG. 8. ORIGINAL DESIGN OF DISCHARGE CHANNEL

be carried out in summer temperatures. Each length has a 3-ft. cutoff collar at one end, integrally formed. Larger collars (6 to 8 ft. wide radially) will be placed at the center of the dam.

The water spouting from the conduits at great velocity (see Table 1) must be brought down to not over 6 to 8 ft. per sec. before it can be discharged into a natural channel. The assumption was made that 12 ft. per sec. is a maximum for flow over riprap. Of the possible means for destroying the energy stored in the spouting water and bringing its velocity down to the limit indicated, the hydraulic jump or standing wave was chosen as best. It was known that the jump would result automatically in any event and would be fairly straight and regular if the discharge were flattened down to a thin, wide sheet. But unusual conditions had to be provided for, in the form of very high initial velocity at full head in the basin, variable velocity during filling or emptying of the basin, and concentration of the discharge in several separated jets—one from each conduit. Under these conditions there might be such irregularity in the line of the jump and such variation in its position as to require an unduly long concrete channel, which was bound to be very costly. Experiments were undertaken to develop a form of channel that would regularize and stabilize the jump, so that a short discharge channel would suffice. Two long series

of experiments were made, on a linear scale of one-sixteenth and a velocity scale of one-fourth full size (one-fourth because $\frac{1}{4} v$ is proportional to $\sqrt{\frac{1}{16} h}$). They were made on the farm of E. A. Deeds, chairman of the Board of Directors of the Conservancy District. A small lake furnished convenient opportunity for water-supply and discharge. A large centrifugal pump, forcing water through two pipes about 6x9 in. each, and a wooden flume 10 ft. wide by 26 ft. long by 3 ft. deep, within which various forms of wooden discharge channel could be built up, constituted the apparatus (see Fig. 1).

The first series showed that a 10% down-slope in the channel, following a smoothly curved widening, would develop a good uniform sheet, near the foot of which the jump would occur. The discharge channel designed in the early part of 1915, shown by Fig. 8, was based on these experiments.

JUMP STABILIZED BY STEPS AND TAILWATER WEIRS

Still greater stability and regularity seemed desirable, however, especially to avoid local high-velocity currents in part of the width at the delivery end of the channel. An elaborate set of experiments on the effect of obstructions in the channel was therefore undertaken. It was believed that such obstructions would localize the jump in reliable manner. The belief was verified by the experiments.

The two principal elements of variation tried out in these experiments were: (1) Weirs in the channel just beyond the jump, to control the level of tailwater; (2) irregularities or roughnesses in the bottom of the sloped

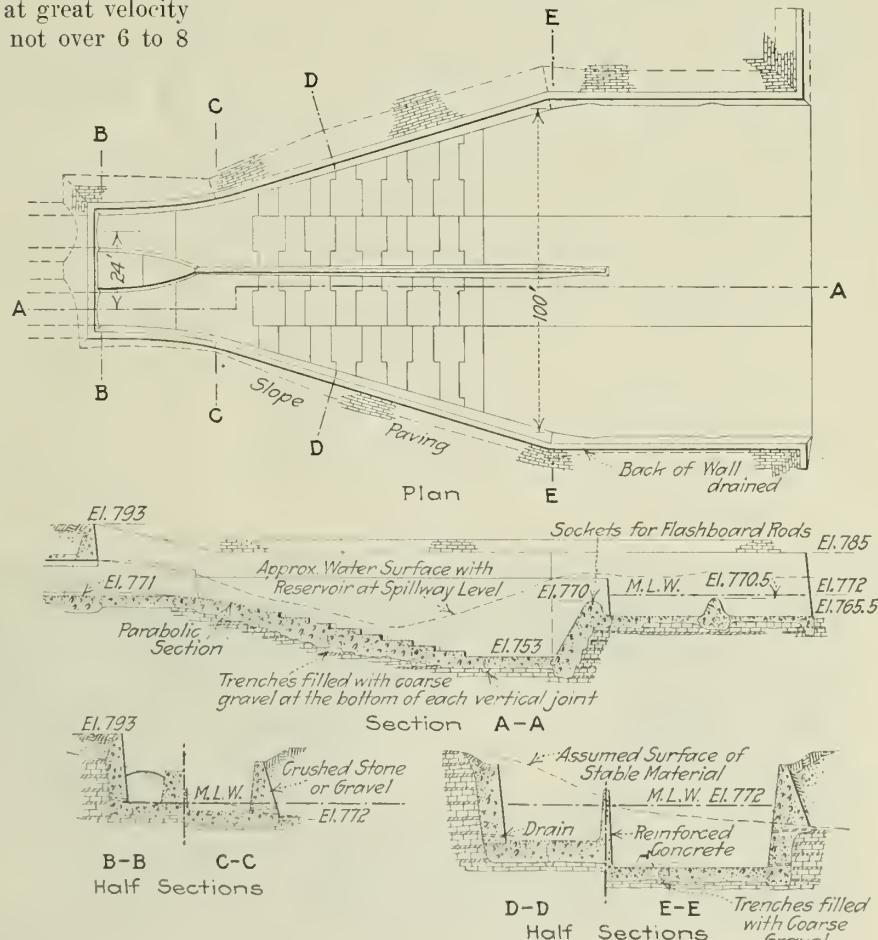


FIG. 9. FINAL DESIGN OF DISCHARGE CHANNEL

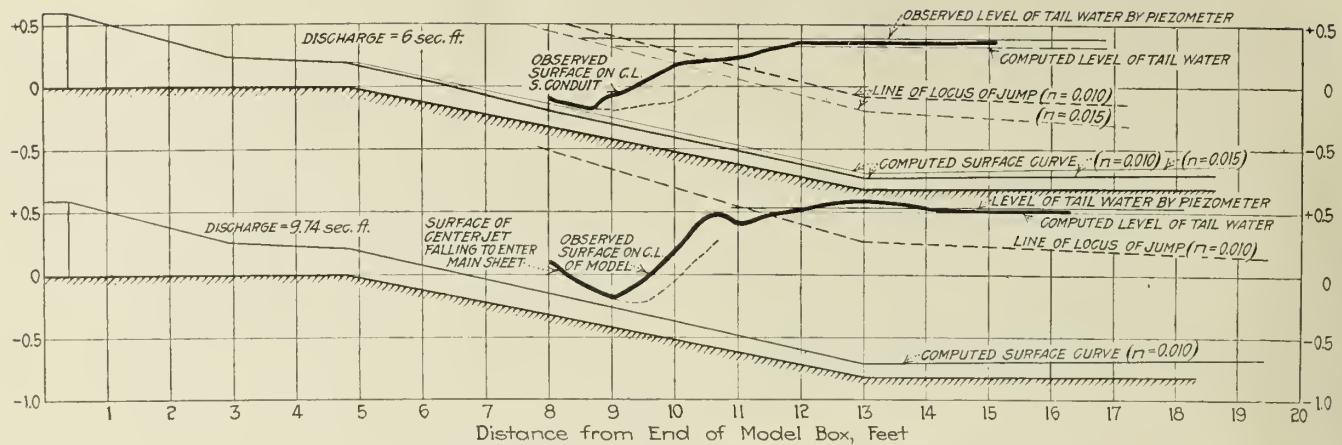


FIG. 10. PROFILE OF JUMP, FROM ONE OF THE EXPERIMENTS

channel, to produce more equal lateral distribution of velocities in the water approaching the jump; and (3) division walls to separate the several conduit jets, extending for various distances, to a maximum where the end was well beyond the jump.

It was early found that a 15% slope in the channel was better than a 10% slope. Some of the further conclusions from the experiments are as follows:

The jump begins above the computed position and ends below it, the latter point being indefinite.

When the water can be made to enter the jump in a sheet of uniform thickness and velocity across the channel, the jump is stable; that is, it is uniform across the channel, and the tailwater moves away without great excess of velocity at any point. The side wall must not be concave toward the water until the jump is passed.

A nonuniform sheet tends to produce an unstable condition of the jump, causing the flow below to be concentrated at high velocity on one side of the channel. Stability is secured by the use of one or more submerged weirs below the jump. The weirs must not be placed close enough to the jump to interfere with its normal formation.

When the flow from one conduit is materially diminished, a high weir is necessary, the most satisfactory height being about one-half the depth of the tailwater at maximum discharge. The addition of a second weir below tends to increase the stability of the jump and to improve the distribution of flow below.

The inclined portion of the channel above the jump may be shortened and steepened indefinitely, provided that the floor be not below the parabolic path of the water issuing as a jet and that sufficient length be provided to permit the water to spread so as to develop a thin and nearly uniform sheet.

Roughening the inclined portion of the channel floor with steps or other equivalent means checks the velocity by increased friction and increases the stability of the jump.

Three views of the jump as realized in the experiments are given in Figs. 11 to 13. A sketch of the profile of the jump in two cases is reproduced in Fig. 10.

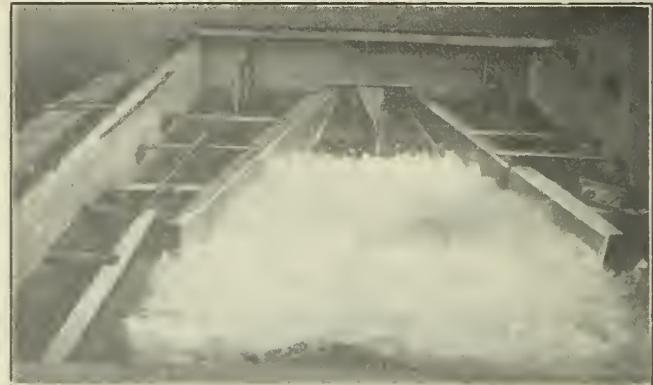


FIG. 13. JUMP WITH 6-IN. WEIR 1 FT. BELOW FOOT OF SLOPE; DISCHARGE 9.74 SEC.-FT.

These experiments furnished the basis for the complete detailed design of the discharge channel. The final result is given by the drawing, Fig. 9. It includes the stepped slope, the pair of submerged weirs and the dividing wall to a point beyond the jump.

The discharge channels will all be founded directly on rock (Cincinnati formation), and will have a substantial thickness of concrete floor. It will be necessary to construct this floor in separate blocks, on account of its size; but care will be taken to avoid any large grooves or openings between the blocks, where the high-velocity eddies of the discharge might get a hold for destructive action.

Protecting the conduit entrances against drift was not as serious as might be supposed without an analysis of the situation. Floods large enough to pick up any drift of



FIGS. 11 AND 12. SMOOTH CHANNEL AND SYMMETRICAL FLOW; DISCHARGE 9.74 SEC.-FT.

troublesome size would quickly submerge the conduit openings. Drift that enters the basin or is picked up by the rising waters after a small pond had been formed would be moved only very slowly by the current, but its movement would be controlled largely by the wind. Even should it reach the vicinity of the conduit inlet, it would

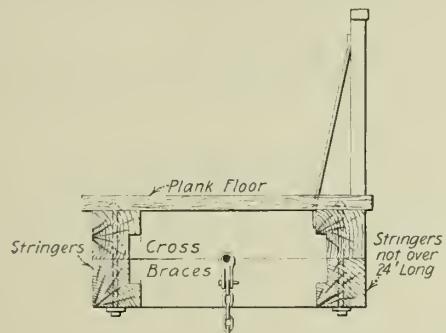


FIG. 14. SECTION OF FLOATING BOOM FOR DRIFT

either pass straight through or float harmlessly on the surface. With the designs of inlet adopted the possibility of clogging would be exceedingly remote.

Because of popular doubts on the subject, however, and also because drift barriers could be constructed at small cost, it was decided to provide further protection by a series of concrete piers spaced 10 ft. apart in the clear, rising nearly to spillway-crest level, and a floating boom. Large pieces of drift—trees, telegraph poles or even

houses—would be held back by the piers. Smaller floating material would be held back by the boom.

The boom is anchored at intervals to heavy abutment piers and normally rests on top of the intermediate piers. The boom design reproduced by Fig. 14 was included in the Official Plan, but this may be improved before construction is started; it is regarded as adequate, but not necessarily the most practical form of construction.

The boom is normally about 30 ft. above top of conduits. Its channel portion hangs rather low and may be within reach of one-year or two-year floods, although these are expected to go through most of the basins without detention.

A considerable number of engineers exercised independent responsibility in working out the details of different portions of the designs. The final results were brought together under the general supervision of Charles H. Paul, Assistant Chief Engineer, and S. M. Woodward, Consulting Engineer. J. H. Kimball had charge of channel improvements throughout the ten cities and towns involved. Walter M. Smith, Designing Engineer, had charge of design of dams and related structures. R. M. Riegel worked out the experimental investigation of the hydraulic jump and the details of design of the combined outlet conduits and spillways. O. N. Floyd had charge of plans for railroad relocations and changes to other public utilities. H. S. R. McCurdy made studies of subsurface conditions and conducted investigations and tests of materials available for foundations, embankments, etc.

The Miami Valley Flood-Protection Work

IV—Study of Retarding-Basin Operation

Necessity of Predetermining Frequency and Extent of Basin Flooding; Methods of Calculation of Storage for Known Floods; Flood "Routing" for Past Quarter-Century; Quick Recovery of Basin System; No Summer Flooding

The five great retarding basins of the Miami Conservancy District, though designed with sole regard to their service in the largest possible floods, will also operate in other floods. Moderate floods are relatively frequent; and therefore, though their effects are of less amount, they have a vital bearing on the operating value of the project.

It has been stated, with truth, that a flood little more than half the volume of that of 1913 would do practically the same amount of damage in the Miami Valley cities. A similar statement holds for Columbus, where the Scioto flood of 1916, less than one-third as large as the 1913 flood, came close to the levee tops—and with a little additional volume would have again swept the west side disastrously. Under these conditions the certainty and degree of control which the basins exert on such moderate floods must be known, especially with a view to estimating the relative benefits which different parts of the territory protected only by the retarding basins will receive.

Primary questions on the functioning of the system are such as these: What effect will the basins have on a flood like that of 1913? How large will be the river flow at different points, and where (if anywhere) will the water go out of its banks? How large an area in each of the several basins will be flooded, and how long will water remain on the different parts of this area?

Similar questions asked with respect to floods of a magnitude recurring once in ten years, or once in five years, on the average, are still more important. The answers will determine, for example, what use can be made of the 35,000 acres of good farm land embraced in the basins. If the average annual flood submerges, say, 3000 acres of

this territory, and if this is more likely than not to happen in the growing season, then the 3000 acres will be idle land, of little value.

Even questions of strictly engineering character, concerning planning of the works, are involved. In order to compare the cost of different plans of flood control, consideration must be given to the relative losses in land rendered unfillable by the frequency of flooding. So also

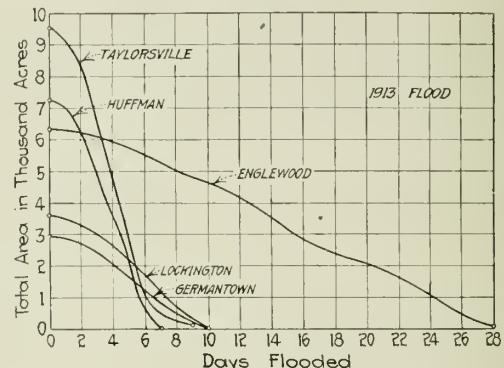
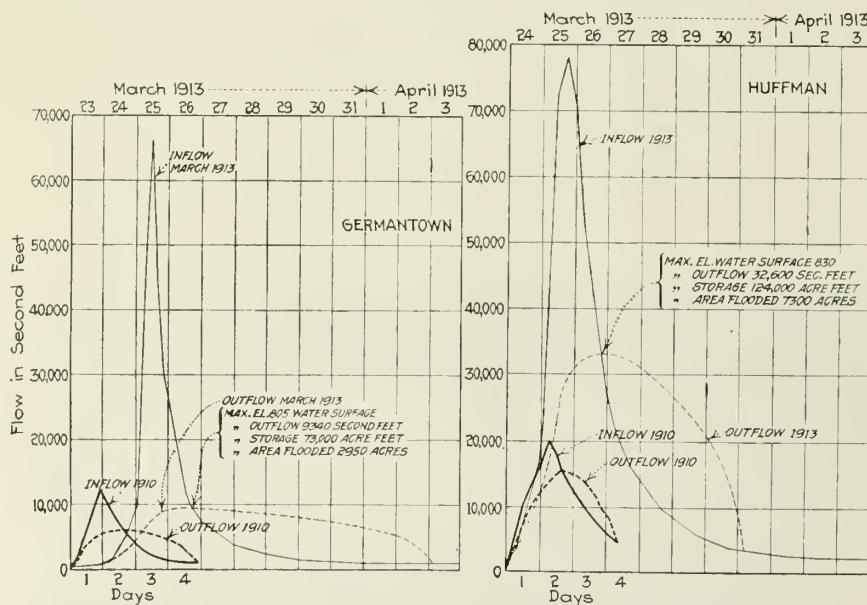


FIG. 3. CURVES OF AREA AND DURATION OF FLOODING IN THE BASINS FOR FLOOD LIKE 1913

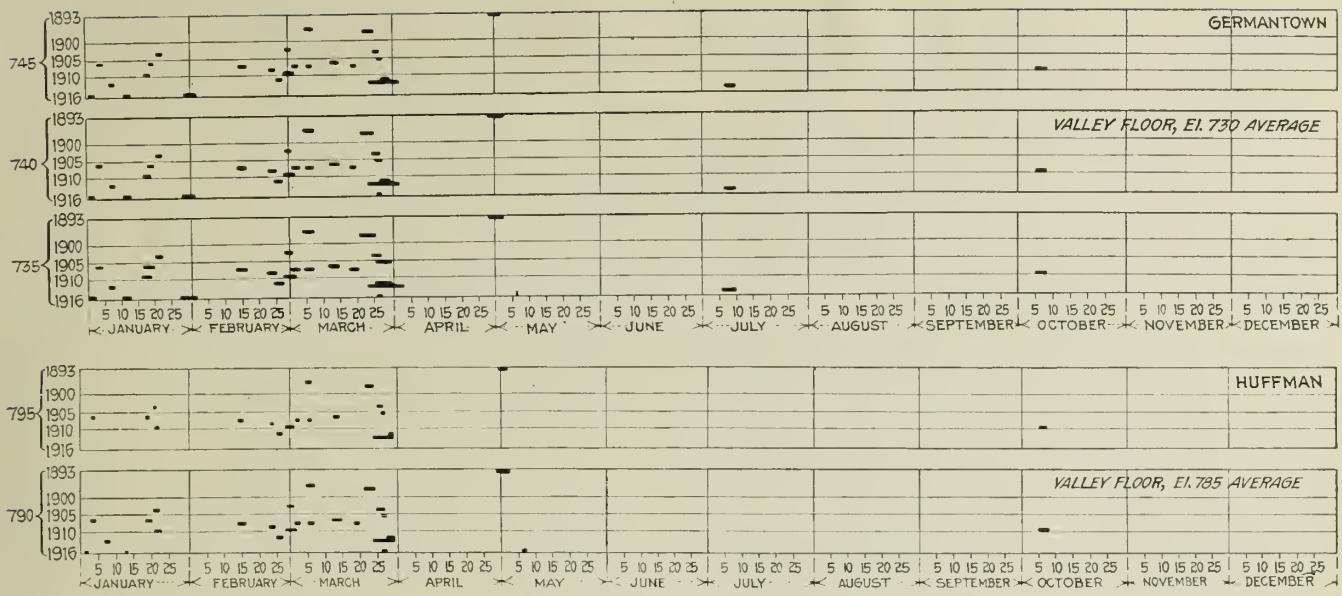
the value of a plan will depend partly on the degree of protection from annual floods given to agricultural lands in the valley bottoms below the basins. The height to which ordinary floods will rise in the retarding basins may affect the location and height of drift-catching devices, and may have some influence on the necessity for railway and road relocations. The "recovery" of the flood-control works after a flood—that is, the quickness with which the basins

empty themselves and return to normal conditions—is an important factor in defining the security of the system in case of floods in sequence. These questions were answered by a graphical study of basin operation for all the floods whose data are fairly well known. Records were found for all the floods of 24 years, 1893 to 1916, and the study was carried through for each of them. The flow curve of each flood at various points along the river was known, or could be approximated from the data of the river stage records. This curve, for a point at one of the retarding basins, represented the inflow into the basin. In the case of the Taylorsville basin, which is affected by the Lockington basin above, the inflow curve had to be drawn by first finding the Lockington outflow and adding to it the normal (unretarded) flow from that part of the drainage area which lies below Lockington. The rate of outflow from the basin in a flood depends on



FIGS. 1 AND 2. CONTROL CURVES FOR GERMANTOWN AND HUFFMAN RETARDING BASINS FOR 1910 AND 1913 FLOODS

Light lines show flood of March, 1913, heavy lines show flood of October, 1910. The inflow represents the magnitude of the flood without retardation, while the outflow represents the size to which the flood would have been reduced if the retarding basins had been in existence



FIGS. 4 AND 5. TOTAL FLOODING DIAGRAMS FOR 24 YEARS, 1893-1916, HUFFMAN AND GERMANTOWN BASINS

the height to which the basin is flooded. This height in turn depends on the volume stored, which is the difference between the inflow and the outflow. Thus the quantities involved are inter-related. The problem of computing the rate of outflow had to be solved by trial and error.

The calculation was carried out in the form of a tabulation containing in parallel columns the values of inflow rate, outflow rate, storage increment, depth of water and area flooded, for successive time intervals. The inflow was read directly from the flow curve. Assuming then a certain rate of outflow, the difference between inflow and outflow gave the rate of storage, which multiplied by the time interval gave the storage increment. Adding this to the previous storage, if any, gave total storage. There was available a previously computed table for each basin, giving for every foot of storage depth the quantity stored and the area of water surface, all figured from the topographical map of the basin. Using this table, the depth of water which the total storage at the given time would produce was read, and by referring to discharge-rate values calculated from the conduit characteristics the conduit outflow for this depth became known. If the outflow rate so determined checked with the assumed rate of outflow, the computation verified itself.

When these operations had been carried through for the whole series of time intervals covered by the inflow curve, the result was a table of outflow and storage, from which all further questions could be answered.

BASIN-CONTROL CURVES

Laying off the inflow and outflow rates on a time base to form a time-flow diagram, curves were obtained that show graphically the effect of the basin in controlling (reducing) the particular flood in question, at points immediately below the basin. Such curves are reproduced in Fig. 1 for Germantown basin and in Fig. 2 for Huffman basin. Both diagrams represent two floods out of the 28 of the past 24 years. The March, 1913, flood is shown because of its great magnitude, while the October, 1910, flood is shown as a typical dry-weather flood—one whose effects might be important with respect to farm operations.

It will be seen that the outflow curve reaches a maximum where it intersects the inflow curve. The area be-

tween the two curves at the left of the intersection represents the maximum storage. How the basin lowers the flood peak and flattens it out is obvious at first glance. The outflow curve is the reduced river flow below the basin. By adding to it the tributary flow coming into the river at points farther down, a complete picture of the controlled flow is obtained, and it is then possible to make calculations of overflow, required channel enlargement, etc.

DIAGRAMS OF BASIN FLOODING

The table used in plotting the control curve also furnished data for a curve of area of basin lands flooded. Such curves were first plotted on the same time base as the control curve and then, by using the horizontal intercepts of the first curve as abscissas of a second, the relation between area flooded and number of days' duration of flooding was represented graphically.

The duration-of-flooding curves for all five basins for the 1913 flood are grouped in Fig. 3. They bring out the fact that the retarding influence of the basins upon this particular flood ranges from 7 days to 28 days. Englewood basin has by far the longest service, while the other four have nearly equal times of emptying. It is interesting to observe, however, that the two largest of these four basins empty most rapidly, although at peak storage they flood the most land. The relations shown by these curves are not the same for different floods, however. A single diagram like Fig. 3 is therefore not a safe basis for generalizations.

The area-time curve, from which the duration-of-flooding diagram was derived, furnished the data also for an important chart quite different in character. This is the frequency-of-flooding diagram, or, as it was named from its appearance, the pianola diagram (Figs. 4 and 5).

FREQUENCY OF FLOODING DURING PAST 24 YEARS

Nearly 40,000 acres of land, of which a large part is excellent farming land, lie back of the dams below their spillway levels. This land is worth probably from \$3,000,000 to \$5,000,000, and its annual producing power may be placed conservatively at \$500,000. The Miami region ought not to be deprived of this large producing territory or needlessly taxed by so large an annual loss. There was much apprehension in the valley over the effects of the

flood-control project in laying idle these agricultural areas, and it became an important issue in the prolonged Conservancy Court hearing on adoption of the plan.

The Conservancy District intends to buy outright as much of the basin lands below spillway level as may be necessary, in order to eliminate questions over continued occupancy, as well as dispute and litigation over the value of basin easements. However, this does not reduce the necessity for preserving as much as possible of the productive value of the basin lands.

By charting the frequency of flooding in the most comprehensive manner that could be devised a remarkable and in all respects reassuring picture of basin conditions was secured. Figs. 4 and 5 reproduce the charts for Germantown and Huffman basins, the one typifying the basin with relatively small outlets, the other the basin with large outlets (Taylorsville and Huffman). Only the lower stages are covered by the reproduced charts, although the originals were carried up to the maximum storage.

There were only four dry-season floods during the entire 24 years; the fourth is barely visible in the lowest line of the Germantown chart, Fig. 4. During the entire period only two floods occurring in the early growing season would have submerged any material part of the valley bottom. Since these two include the 1913 flood, whose probable mean recurrence interval has been estimated as more than 150 years, the frequency of flooding is even less than the chart indicates. The lowest basin lands will not be submerged on the average oftener than once in 15 years during the months of April to September inclusive. At the higher elevations the flood effects decrease rapidly.

The relatively frequent winter and spring flooding is expected to have no effect on agricultural use of the land, or, if any, a beneficial effect. The opinions of experts and experience in the frequently flooded bottoms near the mouth of the Miami joined in showing that winter or spring flooding is likely to increase the fertility of the land. For this reason the middle portion of the charts, covering the growing months, is most significant.

THE BASIN-RECOVERY CURVE; FLOODS IN SEQUENCE

To determine how soon after a flood the basins would again be in normal condition it is sufficient to refer to the control curves, such as Figs. 1 and 2, from which the time when the control ceases—that is, when storage is discharged—can be read. But the amounts of storage capacity available at various times before the complete emptying are not evident. To show these amounts the curves represented by Fig. 6 were drawn.

These curves, derived from the data given by the control tabulations, show percentage of basin capacity left empty at varying periods of time after peak of flood. Fig. 6 is for the Huffman basin and the 1913 flood, the largest on record. Even at peak storage about 25% of the spillway-level capacity would remain empty. Two days later no less than 60% of the basin capacity would be

available to receive a following flood, and in four days after peak stage the entire capacity would be available.

This study is important mainly because so little experience is at hand concerning the possibility of two flood storms occurring in quick succession. Conceding such possibility for storms less than that of 1913, the duplication of a 1913 storm within a week appears quite impossible. Yet, the curves show, even in such a case the system is safe (and that without trenching on the reserve capacity, above spillway level). In addition, the recovery curves have bearing indirectly on the question of basin functioning in a storm of more than three days' duration.

To summarize in brief a few of the results of this basin study, the following approximate statements may be made: Ordinary summer floods will go through the two large basins (Taylorsville and Huffman) without even filling the conduits and with but momentary wetting of the low bottoms close behind the dams (not over 200 to 400 acres). In the other three basins the summer floods will produce material storage. It follows that ordinary

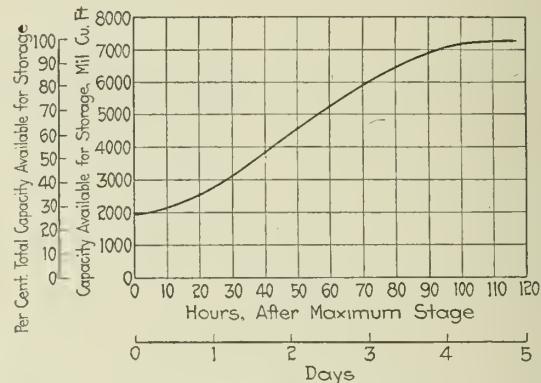


FIG. 6. RECOVERY CURVE FOR HUFFMAN RETARDING BASIN (FOR FLOOD LIKE 1913)

small floods are subjected to but little control at Huffman and Taylorsville.

The average five-year flood (fairly well represented by the 1910 flood, see Figs. 1 and 2) will produce considerable storage in all the basins and therefore will be reduced in stage throughout the valley. The average 25-year flood will be reduced very materially by basin retardation. For both the 5- and the 25-year floods, therefore, farm lands in the valley below, where no channel enlargement is done, will derive benefit from the Conservancy works, while they will get little or no protection against the normal annual flood.

Kenneth C. Grant had charge of studies of action of retarding basins during floods and directed the preparation of practically all the tables and diagrams referred to in this article; A. B. Mayhew made numerous studies of balancing various reservoirs in a harmonious system; Barton M. Jones developed general methods for determining requisite spillway capacities for any set of conditions.

The Miami Valley Flood-Protection Work

V—New Features in the Specifications

Specifications for Contract Work Which Have Been Especially Drawn To Secure Good Contractors and Low Cost; Extensive Departure from Precedent; Clearness and Equity; Engineer's Authority Limited

The contract specifications for the \$20,000,000 Miami flood protection are as strikingly new and individual as the plans for the work. The specifications have been designed, not compiled. They are an instrument devised to handle the varied work of the huge enterprise effectively, to secure the best contractors, to get lowest contract prices and to produce good work without delays or contention.

The preliminary engineering of the Miami Conservancy District had been so directed as to secure a mass of data on rock, soils, water and other conditions. By means of this survey, many questions had been determined in advance that ordinarily must remain unsettled until long after the contractor begins work. This fact, coupled with the desire to make all the advance information available to bidders, exerted a profound influence on the shaping of the specifications.

The engineers have tried to make the specifications clear, thorough, fair and businesslike, and they seem to have succeeded to a remarkable degree. If in the actual administration these specifications bear out their present promise of efficiency, they will have blazed a new trail through the tangle of contract difficulties.

To indicate a few of the prominent features of these specifications: They give a large amount of advance information, tending to reduce the bidder's uncertainties and risks. They make provision for many of the contractor's requirements, such as camp sites, borrow pits and dumps, thereby eliminating further elements of costly uncertainty. They define the contractor's obligations so comprehensively and precisely that at no time is there room for doubt as to what he must do and what he need not do. They set forth the business relations and procedure in definite and satisfactory manner. They bind the owner to as careful observance of terms as they do the contractor. They provide numerous safeguards for the contractor's interests, limiting the arbitrary power of the engineer; but they arm the engineer with great power of decision and compulsion in those matters where the success of the work, or the welfare of the public, may require that he be free to direct, and to enforce his demands. They call for normal quality of work except where specially high quality is needed, but in these latter instances they set forth the required quality in such plain terms as to insure its being obtained.

Points of interest, though they may be secondary, reside also in the simplicity of the language—that is, the elimination of much complex legal phraseology and other heritages of tradition—and in the great reduction of clauses specifying "the engineer's satisfaction." With respect to the latter, a special campaign was made to eliminate the phrases "approved," "satisfactory," "as directed," "to the engineer's satisfaction," etc. One of the final drafts of the specifications was gone over in detail by two members of the engineering organization who are experienced in heavy construction work, and every

instance of these terms was studied to find out whether definite description of a desired result could be substituted. This laborious revision is the more worthy of notice here because its results are likely to be overlooked, since they show only in a negative way.

The specifications are unusually businesslike. In stating time and terms of payment, in specifying results and methods in a measurable way, in providing for successive partial release of surety, in facilitating the adjustment of the final estimate, and in many other ways, their tendency is to establish systematic business relations, largely freed of the personal factor.

A valuable feature is the quick-reference arrangement of the document. It is essentially a self-indexing system, tied and referenced to the unit-price items of the bid.

THE MECHANICAL ARRANGEMENT: A SELF-INDEXING SYSTEM

The specifications have three divisions: General Conditions, General Specifications and Detail Specifications. The latter division, intended as the primary reference portion, is divided into numbered sections, or "items," 1 to 90, which deal with the separate classes of work included in the contract. Each item corresponds to the bid item bearing the same number, as Item 1, Stream Control at Dams; Item 56, Laying Vitrified Round or Channel Pipe; etc.

Whatever part of the work the contractor may be engaged in, knowing the numbered item of the bid schedule under which he is working he has only to look up the detail-specification item bearing the same number. This gives him at once all the necessary engineering and accounting information, a statement of the kind of work covered by the item, directions for doing the work and the conditions of measurement and payment, or a reference to appropriate paragraph numbers of the General Specifications.

As may be understood herefrom, clauses of general applicability to methods are grouped separately to form the division General Specifications. Sections here included are such as Assistance in Making Tests, Protection of Existing Structures, Embankment (six methods of embankment construction are described and specified in minute detail), Concrete (fine aggregate, coarse aggregate, water, forms, mixing, placing, etc.). Similarly, the relations between the two parties and all matters auxiliary to construction, as Camps, Medical Supervision, Extra Work, Hindrances and Delays, Payments, etc., are covered in the division General Conditions. The clauses of the General Conditions are numbered 0.1 to 0.57, and those of the General Specifications 0.57 to 0.105, which successfully avoids all confusion with the item numbers of the Detail Specifications.

Individual specifications for each of the separate contracts in the enterprise are prepared from the main or consolidated specifications by simply omitting item para-

graphs for work not included in the particular contract. The numbering is retained unchanged, so that, whatever the contract may be, Item 5 is "Excavation, Soil Stripping," and Item 37 is "Backfilling." This expedient is an important safeguard against mistake, in dealing with a large number of contracts.

With regard to the element of fairness, it is pertinent to state that this was considered vital. Many clauses virtually specify the engineer's fairness. Thus, in the clause empowering the engineer to direct the manner of conducting the work, an added phrase, "in so far as may be necessary to secure the safe and proper progress and quality of the work," was introduced for the purpose of putting the burden of proof on the engineer and limiting his right to dictate. And under the same head the responsibility for preventing interferences between different contractors is placed on the engineer in express terms. Where inspection of work involves damage to such work, the contractor is to be compensated for this damage unless the inspection reveals bad work.

That a great deal of arbitrary power is yet left to the engineer, notwithstanding this desire to be wholly equitable, is to be explained by the possibility that contracts may have to be let to the lowest bidder, according to the underlying law, unless the Conservancy Court sanctions another award.

The district undertakes to furnish the contractor all right-of-way, all camp sites at dams, and ground for borrow and dumping. In consequence it has been possible to state many requirements more specifically than is usual in contracts, and on the other hand certain provisos had to be made for failure of the district to furnish these facilities promptly.

DEFINITELY LIMITED AUTHORIZATION FOR EXTRA WORK

Carefully drawn safeguards surround the subject of extra work. These act as a check against a contractor's possible inclination to regard every instruction given him as cause for a bill for extras. On the other hand, equal care has been taken to point out specifically those cases where a claim of compensation for extra work is admissible. Thus, failure to furnish right-of-way calls for compensation:

In case of serious delay and loss to the contractor because of failure of the district to furnish right-of-way which in the opinion of the engineer is necessary to the work, the contractor shall be compensated for such loss.

Similarly, when work is stopped by the engineer's direction for any cause other than is specifically authorized in the contract, compensation for extra expense is to be paid.

In general it is provided that extra work (to be ordered in writing by the engineer) shall be paid at actual cost plus 15%, the cost being without overhead. However, if the engineer orders change or increase of plant to promote progress, it is to be considered as regular work done under the contract. Similarly, specific exception is made of night work. Provision is made for claims for damage caused by acts or omissions of the engineer.

In contrast with the individuality of the extra-work clause is the way in which the related matter of changes in plan is disposed of. The right is reserved to the Board of Directors to make such changes as it may elect "in the line, grade, form, location, dimensions, plan or material" of any part. The contractor has no claim (beyond payment at the bid unit prices) except for work

done or material furnished that is rendered useless by the change. There is no clause in this section limiting the amount of reduction without compensation for profits, but in this connection the form of bond provides that no alteration shall be made, without the consent of the surety, which will "alter the general character of the work as a whole, or . . . increase the total amount to be paid to the contractor by more than 25%."

HOW THE TIME QUESTION IS HANDLED

The contract lays special stress on time of completion. It segregates from the contract time an initial period agreed upon as the "period of preparation," during which the plant is to be assembled and working forces are to be organized. In computing rate of progress, seasonal weather conditions are taken into account by rating as two working months the winter period from Dec. 1 to Mar. 31, while the remaining calendar months are rated as equivalent to ten working months. The expected winter rate is therefore only two-fifths of the expected summer rate, as measured in ordinary working days.

The contract fixes as liquidated damages for noncompletion at the specified time (or as it may be altered by extensions) a sum made up of two parts, one sufficient to compensate for the expense of employing engineers and inspectors, etc., and the other representing for each month $\frac{1}{2}\%$ on all money paid to the contractor. These provisions, which put the matter of damages on a mathematical, wholly defensible basis, are considered so fundamentally important that they are embodied in the "agreement," while the further clauses bearing on time are in the specifications.

The contractor must take note of all data and information collected by the engineer and on file for his inspection, and of all adverse conditions; but there is the following important limitation of his responsibility with regard to time:

It is understood, however, that as to river and weather conditions the contractor shall provide against the most adverse conditions and circumstances which reasonably are to be expected to occur on the average within a 15-year period as shown by past records.

The extensive rainfall and flood researches of the district (see *Engineering News*, Jan. 4, 1917) afford specific information on the magnitude of the average 15-year flood and rainfall.

Extensions of time are specifically declared to be possible, in several different clauses, for delay by legal injunctions (if this unavoidably delays the entire contract and not merely parts thereof), for failure of the district to furnish right-of-way and for other similar reasons.

MEASUREMENT AND PAYMENT DEFINED IN PRECISE AND EQUITABLE MANNER

A novel stipulation applies generally to all measurement:

Unless specifically so stated in detail in the contract or specifications, no extra measurements, or measurements according to local custom of any kind, shall be allowed in measuring the work under this contract; but only the length, area, solid contents, number, weight or time in standard units, as the case may be, shall be considered.

It is stipulated and agreed that the planimeter shall be considered an instrument of precision adapted to the measurement of areas.

Two interesting pieces of extra allowance of earth-work are in the cutoff trenches and in road construction:

6.2 Maintenance During Construction—Should material 7.2 wash into a cutoff trench before the placing of em-

8.2 bankment, or stony gullies due to wash be formed, 6a.2 such material and stones shall be carefully removed, 7a.2 if directed, and the contractor shall be paid the contract 8a.2 price per cubic yard for material so removed.

19.5 Maintenance During Construction—The contractor 20.5 shall keep the surface of the roads constructed under 21.5 this contract in good condition, during the life of the contract. When so ordered, he shall fill any depressions caused by settlement or by wear and tear due to use, and he shall be paid therefor at the unit price stipulated for the respective items entering into such repairs.

A still more important provision along the same line is introduced in connection with river excavation for channel improvement:

9.2 Payment—Payment for these items will be made in 10.2 accordance with Section 0.71 of the specifications, 11.2 entitled "Excavation—Payment," with the following exceptions:

Any material that washes into the cut through no fault of the contractor shall be removed by him at the unit price per yard stipulated for these items.

Some individual points of care to avoid trouble attract notice. Under Borrow Pits, warning is given that borrow will not be estimated as excavation (except in specific cases separately provided for); that pits when abandoned (and there is a similar clause for spoil banks) must be shaped and graded, and, if ordered, topsoiled and grassed; that the shaping is included in the item price, but topsoiling and grassing is paid for as a separate item. Excavation of test pits will be credited as extra work. Controlling any springs on the site of an embankment will be paid for, as regards material used, and excavation.

Doubt as to what is included in a given payment item is dispelled by such clauses as the following for concrete:

0.104 Concrete, Prices To Include—Except for cement, reinforcing steel and other metalwork, as provided elsewhere, the unit prices for the different items of concrete shall cover the entire cost of construction thereof, which shall include the cost of materials for fine and coarse aggregates, and of large stones; the cost of all mixing, conveying, placing of concrete and large stones; the cost of damming, bulkheading, pumping, bailing; the cost of preparing foundations, piping or otherwise caring for water, and removing laitance; the cost of grout wash or mortar beds used in building upon old work or on rock, and of coatings for contraction joints; the cost of protecting concrete against frost, floods and injury from other sources; the cost of erecting and removing forms, centers, molds, bracing, landing platforms and scaffolds, and of the lumber, steel, nails, wire and other material used therein; the cost of finishing, patching and wetting; the cost of removing and replacing defective work; and any other work and materials necessary to complete the concrete construction as required by this contract.

But such unit prices shall not include the cost of furnishing cement, the cost of furnishing and placing reinforcing steel, piping, conduits, structural steel, castings and other metal work, which shall be paid for under the items provided therefor.

Even more instructive is the manner in which payment for steel reinforcement is handled:

51.3 Payment—The quantity of reinforcing steel to be paid for shall be the number of pounds placed in accordance with the drawings or orders. The unit price stipulated for this item shall include the cost of metal, the royalty, if any, the cost of transporting, cutting, bending, placing, fastening in position, cleaning, protecting and all other labor and materials connected therewith. Waste material, due to the fact that the lengths furnished are longer than necessary, and wires, clips or other devices used for securing the metal shall not be included for payment. The quantity paid for shall include, however, extra metal in authorized laps wherever the lengths required would be unreasonably long for single bars. In determining quantities, except as the engineer may require test weighing on scales, commercial unit weights shall be used.

As already indicated, the contractor must take all risks, including the hazard of weather and flood:

The risks and uncertainties in connection with the work are assumed by the contractor as a part of this contract, and

are compensated for in the contract price for the work. The contractor, except as otherwise definitely specified in this contract, shall bear all loss . . . from the action of the elements . . . or from any unexpected conditions.

Damage such as flooding of lands above the permanent dams, however, does not fall on him.

PROVIDING FOR HAZARDS AND CONTINGENCIES

With respect to the riskiest item of weather hazard—namely, that involved in stream control during construction of the dams—the above general clause is reinforced by an unusually specific statement. Payment for stream control is a lump-sum item, plus payment for certain materials by measurement (excavation, embankment, riprap). What this payment covers is set forth very broadly in Section 1.3:

The lump sum . . . shall be considered full compensation for all services, labor, material and equipment and for all the resources of the contractor used for or incidental to the operations under this item and for all risks and damages connected therewith.

Hazards that threaten not the contractor but private parties, are dealt with in a separate clause, relating to emergency work, whose opening paragraph reads:

It is understood by all parties to this contract that unusual conditions may arise on the work which will require that immediate and unusual provisions be made to protect the public from danger of loss or damage due directly or indirectly to the prosecution of the work, and that it is part of the service required of the contractor to make such provisions. The contractor shall use such foresight and shall take such steps and precautions as may be necessary to protect the public from danger of damage or loss of life or property which would result from the interruption of public water-supply or other public service or from the failure of partly completed work.

However, if the engineer finds that sufficient precautions have not been taken, he may direct the work done or have it undertaken by others at the contractor's cost. To protect the contractor against undue charges under this clause, it is provided that the Board of Directors may, for cause, allow him fair compensation.

NEW CLASSIFICATION IN EXCAVATION AND BACKFILL

Excavation is grouped in three classes, depending on material, as has frequently been done in the past. A more precise separation is made in this instance, however, principally because of a material composed of rock interstratified with clay, which occurs in the Miami region. The three classes are defined as follows:

Class 1. Earth—Earth, as a name for excavated material, shall be used to include (except where material is classified as Class 2, Mixed Excavation) all glacial deposit, whether cemented or not, except solid boulders $\frac{1}{2}$ cu.yd. or more in volume; it shall include all alluvial deposits and disintegrated material lying on top of ledge rock and all material of whatever nature not properly included under Class 2 or 3. It shall not include the Cincinnati formation . . . earth or any material which has been classified on the drawings as Class 2, Mixed Excavation.

Class 2. Mixed Excavation—The term Mixed Excavation, as a name for excavated material, shall be used to include:

The material known geologically as the Cincinnati formation, which lies under the glacial deposits and consists of thin, approximately horizontal layers of limestone, from a fraction of an inch to a few inches thick, alternated with and embedded in clay, and which may or may not require blasting for its removal; and

Mixtures of any or all materials which, in the opinion of the engineer, it is impracticable to classify as Class 1, Earth, or Class 3, Rock, either because of the character of the material or because of the method used in its removal.

Class 3. Rock—Rock, as a name for excavated material, shall be used to include the solid ledge-rock formation, whenever found (except when classified as Class 2, Mixed Excavation), which is pre-glacial and which can be removed properly

only by means of explosives, barring or wedging or by some other recognized method of quarrying solid rock. It shall also include existing concrete, or masonry with mortar joints, which may be removed under instructions. It shall not include thin layers of limestone embedded in clay, nor cemented gravel, nor any material classified as Class 2, Mixed Excavation.

A distinction is made in the bid schedule and in the items of the Detail Specifications between excavation below mean low water and above this elevation. Items below mean low water (when this is indicated on the plans) are distinguished from those above that level by the suffix *a*; thus, Item 13 is Earth Excavation for Outlet Work, etc., at Dams, but when this excavation is below indicated low water it comes under Item 13a.

For purposes of bidding, excavation is divided into 10 items, differing in location and purpose, which with the above classifications give ample room for varying the price according to cost.

BACKFILL DISTINGUISHED FROM REFILL—The specifications state that excavation includes making the excavation and refilling those portions that may be required. But this refers only to ordinary filling. Puddled or otherwise consolidated refill is designated as backfilling, a separate item in the bid schedule, and as described is to be done by puddling or by rolling or tamping in 6-in. layers.

EMBANKMENT MATERIALS AND METHODS

Six classes of embankment are described, covering the entire range of kind of service and method of construction. They refer respectively to the hydraulic (monitor-and-pump) method; the semihydraulic method (bringing material in by mechanical means, but sluicing it to place by hydraulic streams); the sprinkling and rolling method; and three grades of levee and road embankment. In specifying the material for the dam embankments, there are interesting points in a portion of the controlling clause:

Where Class A, Class B or Class C embankment is specified, it is the intention that the outer portion of the dam shall be composed of coarse material which shall be permanently stable under a condition of complete saturation. Any material which has a tendency to slough, slide or wash when subjected to any condition which may reasonably be expected to occur will not be acceptable. The center portion of the dam shall be composed of material containing sufficient clay or other finely divided particles to be highly impervious to water. To obtain these results the contractor shall use only carefully selected materials. Where the material from one borrow pit does not contain the requisite coarse and fine particles in the desired proportions, other sources shall be utilized; and if necessary to obtain satisfactory results, more than one class of material shall be deposited at the same time in the same or different portions of the embankment.

The preparation of rock and earth foundations for masonry structures involves a suggestion as to what tools should be used, which well indicates the expected result:

The surface shall be scrupulously freed from all dirt, gravel, scale, loose fragments or other objectionable substances. Streams of steam, air or water under sufficient pressure, and wire brushes or other effective means, shall be used to accomplish this cleaning. Steam jets or hot water shall be used to thoroughly remove snow, ice or frost, if any be found upon the foundation when it is desired to lay concrete.

WARNING AND ENCOURAGEMENT AS TO USE OF LOCAL MATERIALS

Unusual care is taken in giving the bidder and contractor advance information about the probable availability of certain materials. Thus, in a well-drawn paragraph on riprap there is contained a distinct warning against the use of most of the local stone.

58.1 **Description**—Riprap of required thickness and composed of tough, durable stones of the size to be prescribed shall be placed as designated in the drawings or ordered. Stones placed on or in the slopes of embankments, as a result of the grading of embankment material, will not be classed as riprap. The contractor will not be required to place riprap by hand, with the exception of such handwork as is necessary to conform to the specified lines. None of the softer limestone formations found in the locality are acceptable for riprap, and only such of the harder limestone as quarry to acceptable shapes and sizes, and the large boulders, will be satisfactory.

Similar warning is contained in the specification for dry rubble paving. In the specification for concrete aggregate it is stated that, subject to test, the local gravels will be satisfactory for use as coarse aggregate if properly screened and washed, but few or none of the local ledge rocks are satisfactory.

The contractor is not only allowed, but encouraged to use in the construction all material obtained from excavation and the like. *He receives double pay* for such material, once for excavation and once for placing. Thus it will be economical for him to use material from construction rather than to borrow and waste. This clause reads, in part:

0.62 **Materials Obtained from Construction**—Materials from the excavations, or from clearing or stripping or other operations, which are suitable and required, shall be used in the construction, as and where directed or permitted, and shall be paid for according to the class of work in which they are used, in addition to the price paid for original removal, except in the case of embankment material for streets and highways, and of material for refilling excavations, where such refill is not designated to be paid for as backfilling.

He is also specifically given permission to enlarge excavations that do not require lining with concrete, in order to obtain material. Such material, however, is not paid for as excavation.

NOVELTIES IN THE CONCRETE SPECIFICATIONS

The engineers considered that final determination of the concrete mixtures would best be made at the time of construction, or at any rate subsequent to the drafting of the specification and receiving of bids. They therefore provided for varying the mixtures over a range generally from 1:2:4 to 1:3:6. This is made equitable by the expedient of paying for the cement as a separate item; the variation in total aggregate (sand + stone) will be immaterial as between richer and leaner mixtures.

For bidding, concrete is grouped in 11 items of different location or kind. The cement requirements are those of the specifications of the American Society for Testing Materials, published in its 1915 Year Book, with the addition, however, that 20% increase in strength must be shown from 7 to 28 days (1:3 briquettes) and that the tests must show reasonable uniformity of the cement. Reinforcing steel is required to be of the structural-steel grade of billet-steel reinforcement as specified by the American Society for Testing Materials, and must be "twisted or otherwise deformed." If it becomes rusty, only "loose rust scale" is required to be removed.

SAND TESTS AND CONCRETE MIXING

Sand for use in concrete must show a 28-day crushing strength at least equal to standard Ottawa sand, in 1:3 mortar. Limestone screenings will not be acceptable as fine aggregate. For stone, no definite tests are prescribed, but an "acceptable" mixture of sizes between $\frac{1}{4}$ in. and $2\frac{1}{2}$ in. must be present. Run of crusher or run of bank shall not be used without screening. Water for concrete

must be "reasonably clean and free from oil, acid, strong alkali or vegetable matter."

A carefully worded requirement is made as to mixing:

0.93 Concrete Mixing—All operations incidental to mixing concrete shall proceed with sufficient dispatch to insure the bonding together of the successive batches as a true monolith. Concrete shall be mixed in approved mechanical batch mixers, in batches of suitable size, except that, when permitted, it may be mixed by hand in a thorough and satisfactory manner. In determining proportions of ingredients, 100 lb. of cement shall be considered 1 cu.ft. The coarse and fine aggregates shall be measured separately and uncompacted, in approved measuring boxes. Suitable means shall be provided for controlling and accurately measuring the water. The entire batch shall remain in the mixer not less than 60 sec., and longer if necessary to secure a thoroughly satisfactory mix.

After requirements for depositing, churning, spading and the usual statement of cold-weather precautions, a severe clause for protection of concrete after placing is inserted:

0.100 Concrete, Protection of—The contractor shall not permit walking or working over or upon finished surfaces of concrete until sufficiently hardened. Concrete shall be kept moist for at least four weeks or until covered with earth, and during this time shall be protected from freezing. Every precaution shall be taken to prevent concrete from drying until there is no danger of cracking or crazing due to lack of moisture. To this end concrete surfaces, unless covered with wet canvas or other equally effective material, shall be sprinkled with sufficient frequency to preclude any possibility of drying out. Alternate drying and wetting shall be particularly guarded against. Great care shall be exercised at all times to prevent injury to concrete surfaces which will be exposed in the finished work. The contractor may at any time prior to the final acceptance of the work be required to clean all exposed faces of concrete.

A very moderate demand for surface finish is put into specific terms, so that the contractor may be sure he will not be led into special expense on this account. Its essential portion calls for simply "removing in an approved manner all the rough edges and projections."

The important case of the interior of the discharge conduits is, however, covered in a very different way. Here an absolutely hard, durable, even and continuous concrete surface must be obtained in order to make sure that the high-velocity flow of water cannot do damage. For this purpose, a very rich mixture using specially graded selected aggregates and molded or finished to a true surface is called for, though the finish may be natural (from the forms):

42.1 Description—The lining of the outlet conduits, except as otherwise indicated in the drawings, shall consist of a special grade of concrete, mixed in proportions, by volume, of approximately 1 of cement, 1½ of sand and 2½ of coarse aggregate, the latter to be composed of fragments of granite, trap or other suitable igneous rock approved by the engineer. Such coarse aggregate shall pass through a screen having 1-in. round holes and be retained on a ¼-in. mesh screen. No large stones shall be placed in this class of concrete. As maximum possible density and exceptionally smooth and uniform interior surfaces are essential for the conduit linings, more than ordinary care shall be exercised in building, finishing and securing forms and in proportioning, mixing and placing the concrete, and the contractor shall employ none but skilled labor for this work. Both the fine and the coarse aggregates shall be especially well graded. To secure the proper gradation it may be necessary to separate the coarse aggregate into not more than three sizes and thoroughly remix in such proportions as directed. After placing in the forms, the concrete shall be so manipulated by spading and joggling as to expel entrained air, not only from the concrete, but also from between the face of concrete and the forms. No plastering or patching of concrete surfaces forming the waterway shall be done unless expressly permitted, and if so permitted shall be done in strict accordance with directions. No thin patching or plastering will be permitted, but recesses shall be cut of such shape as to retain the patches and of such depth as to insure their permanency. If required, anchor bolts shall be set in drilled holes, and these and wire mesh or other suitable device embedded in the patch. The con-

tractor shall supply carborundum brick or emery wheels, if necessary, and shall dress all inequalities on the surface of this concrete.

RIGHT TO RETAIN IMPERFECT WORK

In addition to the usual requirement that the contractor must make good defective work irrespective of prior acceptance of this work or material, there is a clause which gives the engineer the right, under certain circumstances, to retain work found to be imperfect. If the imperfect work is not bad enough to require replacement, it may be retained at a reduced price:

0.8 Retaining Imperfect Work—If the contractor shall execute any part of the work defectively, and if the imperfection in the opinion of the engineer shall not be of such magnitude or importance as to necessitate, or be of such nature as to make impracticable or dangerous or undesirable, the removal and reconstruction of the imperfect part, then the engineer shall, with the written approval of the board, have the right to make such deduction as may be just and reasonable from the amounts due or to become due the contractor, instead of requiring the imperfect part to be removed and reconstructed.

For example, suppose that a contract for one of the dams be entirely completed at the close of a working season, except that the grassing of the slopes under the item Surface Dressing and Grassing had resulted in a poor stand of grass on a portion of the area. To insist that the contractor correct this imperfection would impose a great hardship on him, as it would mean holding the contract open through the winter and for several months into the following working season. Under the clause quoted, the board could make reasonable deduction for the unsatisfactory grassing, pay the final estimate and close up the contract, attending to the reseding later at a suitable season. The value of such a clause, under these circumstances, is obvious. It simplifies the carrying out of businesslike methods in many similar instances.

A FEW ADDITIONAL BUSINESSLIKE PROVISIONS

A remarkable provision that looks to the interests of both parties is made in connection with earthwork for roads. This provision reads as follows:

19.6 Readjusting Grades—Attention is called to the uncertainty of ledge-rock elevations, which, owing to the nature of the work to be done, cannot be determined in advance. In order to avoid shallow rock excavation, or to obtain sufficient suitable materials for embankments and refills, or for any other purposes, the grades shown on the drawings may be changed or modified at any time by the engineer to obtain the desired results.

The intent to facilitate rather than bind and hamper the contractor is seen in the following paragraph:

Wherever in the specifications or in the drawings standard specifications or other publications are referred to, copies thereof may be found, available for reference, at the office of the chief engineer.

Also, in the following:

In the event of any doubt as to the meaning of any portion of the contract, specifications, drawings, supplementary drawings or instructions of the engineer, the same shall be understood to call for the best types of construction, both as to materials and workmanship, which reasonably can be interpreted.

The contractor is also helped by an important provision in the section governing the procedure in making the final payment. Final payment is to be made by the district before the expiration of 30 days from date of final acceptance by the board, irrespective of claims against the district, provided the contractor submits a detailed list and statement concerning any such claims. In other words, the payment is not held up until all claims are

adjusted, but only until data are submitted by which their adjustment can subsequently be taken up.

The contractor receives interest at 6% on all delayed payments, interest beginning on the tenth day after date of estimate, or in case of the final payment, on the thirtieth day after acceptance of the work by the board.

AUTHORS OF THE SPECIFICATIONS

The specifications as finally adopted represent largely the work of Arthur E. Morgan, Chief Engineer, and Charles H. Paul, Assistant Chief Engineer; H. S. R. McCurdy also had an active part in their preparation; but the unusual variety of previous experience represented by the large number of engineers engaged upon this project enabled special first-hand knowledge to be utilized on all sections of the specifications. Among those contributing in an important way to the final result are Daniel W. Mead, consulting engineer; J. H. Kimball, O. N. Floyd, G. H. Matthes, Walter M. Smith and A. B. Mayhew. The specifications and forms were searchingly reviewed from a legal viewpoint by the attorneys for the district, E. J. B. Schubring, of Madison, Wis., O. B. Brown and John A. McMahon, of Dayton, Ohio.

EVOLUTION OF THE FLOOD-CONTROL PROJECT

Immediately after the disastrous flood of March, 1913, a Flood Prevention Committee composed of leading citizens of Dayton, Ohio, began a consideration of ways and means of preventing the recurrence of such a disaster. They employed the Morgan Engineering Co., of Memphis, Tenn., to make all necessary engineering investigations and suggest a plan of procedure. Upon the recommendation of this company, a plan of protection for the whole valley was adopted, a new enabling law was passed by the State Legislature, and under this new law the Miami Conservancy district was established in July, 1915, with its office at Dayton. The directors of the district are E. A. Deeds, of Dayton; Henry M. Allen, of Troy, and Gordon S. Rentschler, of Hamilton. The secretary of the district is Ezra M. Kuhns.

Previous to the legal organization of the district much of the preliminary investigation and general planning was completed, and immediately upon its creation the district took over the whole engineering organization and has carried forward the work to date. As soon as the legal procedure required in the levying of the special assessments against the benefited property can be completed it is expected that contracts will be let and the whole project pushed to rapid completion.

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